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Grout Impregnation of Pre-Placed Recycled Concrete Pavement (RCP) for Rapid Repair of Deteriorated Portland Cement Concrete Airfield Pavements

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Abstract: The U.S. military must have the ability to rapidly deploy troops and equipment anywhere in the world as part of a contingency operation. Recent military operations have highlighted the critical need for rapid repair procedures and materials for military use on sub-standard, in-theater airfields. The U.S. Army Engineer Research and Development Center is currently addressing these problems through a 6-year demonstration-based research and development program called JRAC (Joint Rapid Airfield Construction). This study involves the development of a method using rapid setting grouts and recycled concrete pavement (RCP) to repair portland cement concrete pavements. A laboratory study was conducted to evaluate material properties in order to gain an understanding of expected field performance. Eight full-scale repairs were constructed using two rapid setting grouts, two types of equipment, and two concrete slabs. The repairs were successfully trafficked with simulated C-17 aircraft wheel loads to verify the structural capacity and, ultimately, the procedures.

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PREFACE

This report was prepared by Travis A. Mann and was submitted, in December 2006, to the faculty of Mississippi State University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering in the Department of Civil and Environmental Engineering.

Mann is a research civil engineer in the Airfields and Pavements Branch (APB) of the U.S. Army Engineer Research and Development Center (ERDC), Geotechnical and Structures Laboratory (GSL). He served as principal investigator for the laboratory study and field testing reported herein. Contributing authors to this ERDC technical report were Drs. Reed B. Freeman and Gary L. Anderton. Members of Mann's thesis committee, in addition to Drs. Freeman and Anderton, were Drs. Tom White and Isaac L. Howard.

The laboratory study and field testing reported herein were conducted by GSL personnel at ERDC, Vicksburg, MS, under sponsorship of the Joint Rapid Airfield Construction (JRAC) program. The JRAC program is a comprehensive 6-year, demonstration-based research and development program being executed by ERDC during fiscal years 2002–2007. The program is sponsored by Headquarters, U.S. Army Corps of Engineers.

Work was conducted under the supervision of Don R. Alexander, Chief, APB; Dr. Albert J. Bush III, Chief, Engineering Systems and Materials Division; Dr. William P. Grogan, Deputy Director, GSL; and Dr. David W. Pittman, Director, GSL.

COL Richard B. Jenkins was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

CHAPTER I

INTRODUCTION

Background

The future force of the U.S. military must have the ability to quickly and efficiently transport soldiers and equipment anywhere in the world as part of a contingency operation. The U.S. military's current power projection policy requires that future force projection capabilities meet or exceed the following deployment objectives: deploy to a distant theater in 10 days, defeat an enemy within 30 days, and be prepared for another fight within another 30 days. Current sealift capabilities provide little assistance in meeting these objectives, which leaves strategic airlift as the primary means of providing mobility for the future force. Unfortunately, in many areas of the world, the airfield infrastructure is denied by the enemy, severely deteriorated, or simply does not exist. Currently, light/medium engineer units do not have the capability to rapidly upgrade or construct contingency airfields within the required force projection timeline as defined above.

In light of this shortfall, a comprehensive new program was initiated by the U.S. Army Engineer Research and Development Center (ERDC) entitled "Joint Rapid Airfield Construction," or JRAC. The primary objectives of the program are to

(a) optimize site selection, (b) enhance airfield construction productivity, and (c) incorporate advances in rapid soil stabilization. The JRAC program will serve as the vehicle by which military engineers are provided with new tools and methods that will ultimately allow them to construct and/or upgrade contingency airfields to support future force projection operations. The JRAC program will also drastically reduce the logistical footprint required to build or repair contingency airfields by minimizing material and equipment quantities required for construction.

The JRAC program is a comprehensive six year research effort that began in 2002 and includes over 30 individual work units focused on providing engineering solutions to increase the U.S. military's capability to rapidly build or upgrade contingency airfields. The program schedule includes two major technology demonstrations where the tools and techniques are used by engineers in a military exercise environment. The first demonstration took place in 2004 at Fort Bragg, North Carolina, and the second event is scheduled for 2007 in a remote region in Australia's Northern Territory. Although the predominant focus of the program is on semi-prepared (unsurfaced) airfields, significant effort has been placed on solving the problems associated with the rapid repair of existing rigid and flexible pavements which often play a critical role during contingency operations.

Problem Statement

In most rapid deployment scenarios, the first and most attractive option for military planners is to take advantage of existing airfields that may lie within the region of interest. Today, most regions of the world contain numerous paved and unpaved airfields; however the condition and size of these airfields are often inadequate for the large cargo aircraft that are so critical to U.S. contingency operations. Recent operations have shown that typical paved surfaces encountered during contingency operations lack a history of normal preventive maintenance activities necessary to keep a facility in good working order. Most of the facilities are therefore in disrepair before military operations begin. In addition, our large aircraft typically exceed the structural capacity of these airfields, thus greatly contributing to rapid deterioration of the pavement.

In almost all contingency scenarios it is assumed that some type of repairs must be accomplished either to open the airfield to U.S. aircraft traffic or to maintain the facility during continuous contingency operations. These repairs must be completed with very limited equipment and materials and they must also be conducted under a wide range of environmental conditions. Many airfields are located in austere environments where material availability is questionable at best. The ability to access materials may be hampered by security concerns in the region and the quality of the materials available locally may be poor. Because of these issues, the worst case JRAC scenario requires that all construction and repair materials be transported to the site via aircraft, and therefore considerable emphasis has been placed on reducing the amount of materials required.

Due to severe time constraints, material availability, and material quality issues, traditional repair and rehabilitation techniques are usually not appropriate for repairing contingency airfield pavements. Furthermore, traditional techniques are focused on long-term management of the pavement, which also may not be appropriate if the intent is to use the facility for a limited time. Traditional repairs are typically very comprehensive and require specific equipment and materials that are often not present during military contingency operations. Because of these many challenges, there are a limited number of methods and materials that can be used for the rapid repair of contingency airfield surfaces.

The need for materials and equipment to conduct rapid pavement repair, which will minimize disruptions to traffic, has generated a large number of products that are currently available to the highway and airport pavement industry. The U.S. Army Engineer Research and Development Center has recently conducted research relating to the performance of several types of rapid setting materials which are currently used in small quantities to repair spalls on portland cement concrete (PCC) pavements. These efforts were focused on characterizing the materials using laboratory test results as well as field tests to evaluate performance and to provide guidance for engineers in the field on the usage of these products as a spall repair material.

Preliminary results indicated that some of the self-leveling, rapid-setting, cementitious materials are very effective for spall repairs; however, little data exists on the performance of these products when mixed in larger batches and used in full-depth pavement repair. Because of the lack of expedient methods and materials for larger, full-depth repairs, there is considerable interest in developing solutions for this problem which can be quickly adopted by field engineers. This study seeks to develop a solution that is rapid, effective, and logistically attractive. The proposed method involves the use of flowable grouts and recycled concrete pavement (RCP) to accomplish a repair that can be opened to C-17 aircraft traffic in 3 or 24 hours, depending on the material and technique used.

Objective and Scope

The objective of this study was to develop an expedient repair method for portland cement concrete (PCC) airfield pavements using rapid-setting grouts and recycled concrete pavement (RCP). The method involves the excavation of the damaged section using small lightweight equipment and minimum preparation on the base layers under the PCC. Select materials from the excavated concrete are then used as pre-placed aggregates in the repair hole, and the voids are filled with a rapid-setting, flowable grout. The results of this study will be used to provide guidance to military engineers on the use of the proposed method, limitations on materials, and appropriate construction techniques. The investigation included a laboratory study to characterize the properties of the materials and a field study where actual repairs were constructed and validated with simulated traffic for fully loaded C-17 aircraft.

The materials investigated were limited to recycled concrete from a typical airfield pavement and two rapid-setting cementitious materials: Pavemend™ and Type III portland cement grout. The RCP material was obtained from a representative pavement using small, lightweight equipment and was processed to determine its physical characteristics and suitability as an aggregate. Laboratory unconfined compressive tests were conducted on samples of Pavemend™ alone and Pavemend™ impregnated into a representative quantity of RCP. A scoping study was used to develop a suitable mixture design for a Type III portland cement grout with sufficient strength and flowability to penetrate the preplaced RCP. Prior to the field placements, penetration tests were conducted to verify that the grout materials would penetrate the full depth of the constructed repairs. The field study included constructing repairs on two slabs of different thicknesses using the two grouts and two levels of repair preparation. The volume of the constructed repairs ranged from 0.3 to 0.9 m^3 (0.4 to 1.2 yd^3).

CHAPTER II

LITERATURE REVIEW

Research related to the rapid repair of damaged military airfields has an extensive history in the United States. This chapter presents a review of some of the previous work accomplished regarding the materials and procedures associated with the rapid repair of damaged airfield pavements with a focus on those used by the U.S. military.

Methods of Repair

Damage to airfield pavements can occur in a variety of ways. An airfield can be damaged as a result of numerous types of munitions during conflict (battle damage) as well as from typical loading scenarios and environmental factors. Explosions or projectiles which penetrate the pavement may displace or destabilize large volumes of the subgrade material resulting in lengthy repair times. Pavements that have been damaged from loadings or environmental factors can also create significant problems because the distress often involves large areas of pavement surface and can also require a full depth repair.

A significant amount of research was conducted during the period ranging from the 1960s to the early 1990s and focused on the rapid repair of bomb damaged airfields which would have resulted from an armed conflict with one of the Cold War adversaries.

The vulnerability of U.S. airfields in Europe and at home to aerial attacks resulted in large research efforts to develop methods for the rapid repair of bomb damaged pavements. The research efforts, mostly conducted by the U.S. Army Corps of Engineers, focused on methods for backfilling craters, matting systems to serve as foreign object damage (FOD) covers, and also on preparation activities such as stockpiling materials and pre-positioning of equipment which would allow for quick repairs once the bombing was over (Hoff, 1975; Barber, 1983; Vedros and Hammitt, 1985; Hammitt et al., 1986).

In contrast, today's contingency environment is more likely to see military aircraft using existing facilities located in close proximity to the conflict which have been taken for use by coalition forces. These facilities are typically structurally inadequate for our large aircraft and in poor condition due to a lack of proper maintenance. As a result, engineers in the field are spending a significant portion of their time repairing damage resulting from load related distresses as opposed to bomb related pavement damage. TM 5-624 (Headquarters, Departments of the Army, Navy, and Air Force, 1995) provides solutions for repairing deteriorated or damaged PCC pavements; however, these traditional methods involve comprehensive repair techniques and call for a minimum of three days curing time prior to opening the repair to traffic.

Throughout the history of airfield damage repair by the U.S. Military, a number of different distresses and repair types have been identified and categorized based on different factors including geometry, materials used, and structural quality of the repair. UFC 3-270-07 (Unified Facilities Criteria, 2002) divides repairs into two groups including large repairs (small craters, large craters, and camouflets) and small repairs (spalls and other small distresses) as shown in Table 1. Large craters are defined as having an apparent diameter of more than 6 m (20 ft), and a small crater is defined as one where damage extends into the base course and is smaller than 6 m (20 ft) in diameter. Camouflets are craters with relatively small apparent diameters but deep penetration and subsurface voids. Furthermore, spalls are defined as surficial distresses typically occurring at the interface of joints which do not penetrate the full depth of the concrete.

Table 1. Summary of Repair Methods for Large and Small Repairs Per UFC 3-270-07 (Unified Facilities Criteria, 2002)

Distress Type	Cause	Definition	Category	Repair Method	Type of Repair
Large crater	Large ordinance	Apparent diameter > 6 m (20 ft) and damage extends into the subgrade	Large repair	Expedient	Crushed stone or sand grid backfill with FOD cover
Small crater	Medium to large ordinance	Apparent diameter < 6 m (20 ft) and damage extends into the subgrade		Sustainment	Stone and grout
Camouflet	Penetrating ordinance with time delay	Small apparent diameter but deep penetration and subsurface voids		Sustainment/ permanent	Concrete cap
Spall	Small ordinance or environmental related	Surface damage to concrete slab which does not affect structural performance	Small repair	Expedient/ sustainment/ permanent	Partial depth replacement of concrete using a suitable material

In addition to classifying the repair by its size (i.e., large, intermediate, and small) the UFC 3-270-07 (Unified Facilities Criteria, 2002) also classifies repairs based on structural capacity and expected performance. The classifications are related to the expected duration of use for an airfield as follows:

a. *Expedient airfield repair*: Provides an accessible and functional minimum airfield operating surface (MAOS) that will sustain 100 C-17 passes with a gross weight of 227,707 kg (502 kips), or 100 C-130 passes with a gross weight of 79,380 kg (175 kips), or 100 passes of a particular aircraft at its projected mission weight if other than the C-17 or C-130, or the number of passes required to support the initial surge mission aircraft.

b. *Sustainment airfield repair*: Maintains or increases the MAOS to support the operation of 5,000 C-17 passes with a gross weight of 227,707 kg (502 kips), or 5,000 C-130 passes with a gross weight of 79,380 kg (175 kips), or the number of passes required to support mission aircraft at the projected mission weights throughout the anticipated operation, if other than the C-17 or C-130.

c. *Permanent airfield repair*: This repair increases the MAOS to sustain 50,000 or more C-17 passes with a gross weight of 263,008 kg (580 kips), or 50,000 C-130 passes with a gross weight of 79,380 kg (175 kips), or to support a service defined airfield design type, depending upon mission aircraft, in accordance with UFC 3-260-02, Pavement Design for Airfields.

There is a range of repairs between the small and large sizes which can be categorized as “intermediate.” These repairs are not addressed by UFC 3-270-07, but they are addressed in TM 5-624 (Headquarters, Departments of the Army, the Navy, and the Air Force, 1995), which is a technical manual intended more for permanent facilities in the continental U.S. (CONUS) (Table 2). Intermediate repairs will typically be conducted on those localized areas of failed pavement sections that need to be removed and replaced to allow for continued aircraft traffic. Given that TM 5-624 is intended for CONUS facilities, traditional techniques are recommended for intermediate repairs of portland cement concrete (PCC) pavements. These traditional techniques would typically require the failed section to be removed and replaced with a similar section of pavement and require significant curing time before opening the repair to traffic. Thus, the repair scenario that is missing from current guidance is the expedient, intermediate repair (Table 2).

Table 2. Summary of Repair Methods for Intermediate Repairs Adopted From TM 5-624 (Headquarters, Departments of the Army, the Navy, and the Air Force, 1995)

Distress Type	Cause	Definition	Category	Repair Method	Type of Repair
Blow up	Environmental	Environmental or load related distresses that require a full depth repair if the severity level is "high"	Intermediate repair	Expedient	None currently exist (RCP and grout)[a]
Corner break	Load related				
"D" cracking	Environmental			Sustainment	Slab replacement using PCC
Faulting	Environmental/ Load related				
Linear cracking	Load related			Permanent	Slab replacement using PCC
Punch outs	Load related				
Shattered slab	Load related				

[a] TM 5-624 doesn't provide methods for expedient repairs.

The focus of the JRAC program is to provide solutions for an expedient repair although some of the methods may produce results which will be classified as sustainment repairs.

Full Depth or Large Repairs

A series of studies was conducted by the U.S. Army Corps of Engineers during the 1970s and 1980s in an attempt to provide solutions for the rapid repair of bomb damaged airfields. The NATO standard that was current during these studies required that three repairs be conducted in 4 hours. This standard also required that the repairs be able to withstand 16 passes of a 13,154 kg (29 kip) rolling wheel applied through a tire inflated to 2.07 MPa (300 psi) (Hoff, 1975).

Standard procedures for conducting the repairs during this time period called for the crater to be filled with debris blown from the crater (ejecta) and compacted to serve as the subgrade. A select fill aggregate (base course) was then placed and compacted on top of the debris and finally a landing mat system was placed over the base course to serve as a wearing surface. Hoff (1975) describes several problems associated with this technique.

a. It was difficult to sufficiently compact the rubble and debris in the crater,

b. It required a strong cap material (landing mat) which resulted in an elevated section in the pavement that created roughness problems, and

c. The repair team required 121 people to repair three craters and a large majority of those people were needed just for the landing mat assembly.

In an attempt to develop a better procedure, Hoff (1975) approached the problem with a comprehensive set of field trials using regulated-set cement to produce cellular concrete and mortar, which when combined with the ejecta from a bomb crater, would create a permanently repaired runway sub-base and pavement of operational quality within a few hours. The repair would be flush with the surrounding pavement, thereby eliminating the problems associated with anchoring and ramp-up when matting products are used. The technique also had the advantage of working for both small and large craters.

To produce the cellular concrete, various amounts of foam were mixed with regulated-set cement to create concrete densities ranging from 240 to 2,240 kg/m^3 (15 to 140 lb/ft^3). Regulated-set cements typically replace the tricalcium aluminate with calcium fluoroaluminate. The hydration of this ingredient imparts considerable strength to a paste or mortar immediately after it sets and requires a retarder to control set time. The foam was created by using preformed foam, water, and compressed air which was violently mixed in a chamber or nozzle (Hoff, 1975).

Full scale tests were documented by Collum et al. (1978) using regulated-set cement to fill craters. The field tests and procedures were plagued with equipment difficulty rising from the complex mixing and pumping operations required to deliver large quantities of concrete to the repair. In previous studies, the regulated-set cement suffered from extensive drying shrinkage and severe cracking which produced unacceptable results (Hoff, 1975). In the Collum et al. (1978) study, sand was added to the cement to reduce the heat generation, to reduce the amount of cement required, and to provide additional tensile strength to the concrete. Although there was limited success with the repair of some of the craters, the method was deemed too complex and problematic for implementation.

Hammitt (1985) documents the progressive repair of bomb craters in Germany during a comprehensive set of field trials in the 1980s, used primarily as a troop training exercise for the 293rd Engineer Combat Battalion (Heavy). First, the engineer soldiers conducted an expedient repair, then a sustainment repair, and finally a permanent repair which returned the pavement to its original condition. The repairs were trafficked with a modified 4,540 kg (5 ton) truck equipped with an F-4 tire loaded to 13,970 kg (30,800 lb) and a tire pressure of 1.93 MPa (280 psi). The study resulted in some successes; however, there were also numerous problems with equipment, materials, and cold weather resulting in failures in several of the repairs. Problems with frozen aggregate and compaction equipment that could not access all of the lower lifts resulted in lower densities and excessive rutting in almost all of the repairs. Hammitt (1985) also reports a high failure rate when using cold mix asphalt for crater repairs and recommended that it be abandoned as a technique. Hammitt (1985) noted that hot mix asphalt (HMA) and PCC cap techniques were used with a high degree of success.

The Hammitt (1985) study included the testing of two additional techniques that were not part of the unit's standard set of procedures. These methods included repairing a small crater by impregnating a 200 to 300 mm (8 to 12 in.) open-graded stone layer with an epoxy resin product called "SILIKAL" as well as using a stone and grout method with a standard portland cement grout. The SILIKAL product contained a liquid and dry powder component which were mixed with water and then poured into the crater. It was estimated that the product only penetrated an average of 75 mm (3 in.) into the open-graded layer consisting of aggregate ranging from 56 to 75 mm (2.2 to 3.0 in.). All the repairs cracked extensively after five passes of the F-4 wheel load and severe rutting also occurred.

The stone and grout method described in the study (Hammitt, 1985) involved placing grout for half the layer depth and then placing the open-graded stone into the grout and working in the aggregate until it was completely covered with grout. This process was accomplished using a front-end loader and roller until the stone and grout was thoroughly mixed. Additional information on this method is presented later in this chapter. This approach eliminates problems associated with penetration but is a tedious process with large volumes requiring lengthy repair times. The specifics of the grout mixture are not provided in the literature although it is reported that an accelerator was used (calcium chloride) at an unspecified dosage. The stone and grout repairs were trafficked after 12 hours and performed well with only minor spalling occurring in one of the repairs as a result of the top of the concrete cap being frozen.

Barber (1983) summarized much of the work completed by the U.S. Army Corps of Engineers. He affirmed that regulated-set cement provided a less-than-optimum solution. He also summarized subsequent studies conducted at the Waterways Experiment Station (WES) to evaluate several potential alternative solutions. While the results were generally inconclusive, gravity grouts and crushed stone repairs were considered to have merit. The purpose of Barber's (1983) study was to consider new techniques and materials for backfill in craters. In addition, compaction requirements were to be reviewed. Barber's (1983) investigation included new concepts and technologies including earth reinforcements, crater spanning, expanding foams, and roller-compacted concrete. He concluded that compaction requirements could not be reduced and that eliminating the need for compaction by the use of synthetic backfill and other materials such as geosynthetics, was a potential solution to the crater repair problem.

Although a tremendous amount of work was accomplished over the last 35 years, the materials and procedures used to repair large craters have not changed dramatically during that time. Although there are several variants to the approved solution, the UFC 3-270-07 (Unified Facilities Criteria, 2002) still recommends a method in which debris is used to fill the crater followed by the placement of a high-quality crushed aggregate which is then protected with a FOD cover.

Partial Depth or Small Repairs (Spalls)

Spalls are very common, surficial distresses on rigid pavements that typically occur along joints due to slab movement and incompressible material filling the joint (UFC, 2001a). They can also result from small munitions fired at an airfield which do not penetrate into the base course or cause any structural damage.

Most approaches for the repair of spalls involve filling the damaged area with some type of flowable substance which hardens to provide a material that has comparable properties to the surrounding concrete. Spall repair materials are grouped into three broad categories including cementitious, polymeric, and bituminous. Only cementitious and polymeric are currently allowed to be used on military airfields (UFC, 2001a). Shoenberger et al. (2005) provide a more detailed list of some of the common material types (excluding bituminous) that are used for the expedient repair of pavements:

- Magnesium-phosphate cement.
- High-alumina cement.
- Regulated-set portland cement.
- Gypsum cement.
- Special blended cements.
- Type III portland cement with accelerating admixtures.
- Polymer cements.

- Epoxies.
- Methacrylates.
- Polyesters.
- Urethanes.
- Proprietary materials: high waste.

Stroup (1986) also reports on the use of the SILIKAL product mentioned earlier. The German made product was successfully demonstrated for spall repairs although it did have a complicated mixing procedure, was difficult to clean, and contained flammable components. After success with small repair quantities, the material was tested for larger repairs even though there were limitations of mixing large batches. Two mixing techniques were attempted, manual and machine mixed, however both repairs failed after the application of traffic by an F-4 wheel loaded to 11,567 kg (25.5 kips) and 1.97 MPa (286 psi) tire pressure. Severe cracking was observed in both repairs as well as large elastic deformation during trafficking. Likely reasons for the failures include a less than desired grout layer depth caused by impaired penetration of the flowable polymer grout (10 cm instead of the intended 15 cm) as well as dirty aggregate and insufficient void space to allow penetration.

Other methods for spall repair have been investigated, including one detailed by Stroup (1986) and modeled after a British method where a 19.05 mm (3/4 in.) steel plate is simply placed over the spall and a series of “Rawl” bolts are used to secure the plate to the pavement surface. Although the method was successfully demonstrated, it was noted that securing the plate with the bolts was a significant task and a better method was needed to make the procedure time efficient.

Materials for Rapid Repair

The methods used for the rapid repair of airfield pavements previously described in this chapter involve the use of a wide range of construction materials. This section provides additional details on previous investigations using rapid setting grouts and recycled concrete pavement (RCP).

A number of rapid-setting cementitious materials have recently been used successfully for spall repairs on damaged military airfield pavements; however, there can be several problems associated with using rapid setting materials for the larger intermediate-sized repairs. These problems include:

a. The possible need to extend the material with aggregate,
b. Short working times which can be very dependant on temperature, and
c. The possible damage caused by the large amount of heat that is often generated by the cementitious material.

Because the materials must produce strengths very early, they normally have reduced working times compared to normal cements, and hot ambient temperatures can further reduce working times. Problems with heat generation in this case are similar to those associated with mass concrete placements. These problems include maximum concrete temperature, which can affect the ultimate strength gain of the material, as well as maximum temperature differential, which induces thermal stresses within the material that can lead to thermal cracking (ACI, 1987).

Pavemend ™

Pavemend™ is a self-leveling, rapid setting, cementitious material that was developed for a wide range of PCC repairs. It is a magnesium phosphate based cement product (Shoenberger et al., 2005) that consists of nearly 70 percent (by weight) residual materials including, but not limited to: fly ash, crushed glass, volcanic ash, mine tailings, and municipal solid waste ash (Anderson and Riley, 2002). Upon reacting with water, Pavemend™ produces a "chemically bonded composite of inert materials with a mineral structure" (Hyman and Bruce, 2004). Pavemend™ does not include conventional aggregates, but contains very fine grains of metal oxides (Anderson and Riley, 2002). These grains are almost entirely smaller than 0.6 mm (Table 3).

Table 3. Partial Gradations for Pavemend™

Sieve Size	Nominal Particle Size, mm	Percent Passing
No. 4	4.75	100.0
No. 8	2.36	100.0
No. 16	1.18	99.9
No. 30	0.6	98.1

Previously, the Pavemend™ self-leveling material was available in several variations based on the set time of the material. The manufacturer has since expanded the product line to include numerous variations of self-leveling and trowelable mixtures. The Pavemend™ products have been used extensively by the U.S. Military in recent years with mixed results. Previous experience indicates that if the proper mixing procedures are followed and temperature conditions aren't extreme, the material is very predictable and extremely valuable for the rapid repair of PCC airfields due to its allowing the repair to be placed back into service within hours. The mixed results that have occurred in the field, as well as a desire to expand the use of the product, have prompted several investigations into the performance of the material both in the laboratory and in the field.

Lomibao et al. (2004) conducted a study on the effects of temperature, salt water, and gray water on the properties of Pavemend™. Three products were evaluated for this study (PM 5, 15, and 30), and all formulations were mixed with fresh water and salt water at ambient temperatures of 4.4ºC, 22.2ºC, and 32.2ºC (40ºF, 72ºF, and 90ºF). All mixes using fresh water were reported to have good compressive strengths; however, there were problems with short set times for some of the mixes resulting in flash sets. It was reported that the working times can be extended by reducing the critical temperature to which the material is mixed; however, this also results in a lower compressive strength. It was reported that by modifying the manufacturer's mixing procedures, salt water can be used as mixing water to produce reasonable results. However, saltwater appears to make the material even more sensitive to temperature and mixing times and can result in an unpredictable expansion of the material that would result in material failure after a field placement. The mixtures using gray water were somewhat acceptable; however, strengths were lower and the behavior of the material was again difficult to predict.

Anderson and Dover (2002) tested and reported on the permeability of the material compared to that of a typical portland cement grout. Results indicate that the Pavemend™ material is far less permeable than typical portland cement.

Thomas (2004) reports on a study where the Pavemend™ product was evaluated as a potential rapid runway repair (RRR) material for use by the Naval Construction Force (NCF). A concrete mixer was used in this study rather than the manufacturer's recommendation of a mortar mixer. An attempt was made to use the material for large repairs by pouring the material over open graded aggregate. The aggregate was contaminated with dust and fine aggregate during the test, however, so the Pavemend™ material was not able to penetrate the aggregate. After this failed attempt, the material was extended with various sizes of aggregate by adding the aggregates to the mixing chamber. The addition of aggregates was reported as contributing to erratic mixing times. Although limited success was experienced with this approach, repairs of any significant depth could be problematic due to the tendency of the aggregate to "fall" out of the mix and settle on the bottom creating a non-homogenous repair.

Thomas (2004) reports that a second attempt was made to construct a larger repair using Pavemend™ and the open graded stone. In this attempt, clean aggregate was used and the material was poured until it filled the voids of the aggregate matrix and began to pool on the surface. A subsequent investigation revealed that the material penetrated only 38.1 mm (1.5 in.) into the aggregate layer (the total depth of the aggregate layer is not reported). It was also found that a very shallow layer of the hardened surface was easily peeled away with the claws of a framing hammer, indicating that the repair wouldn't sustain heavy aircraft traffic.

During this same investigation, a successful large spall repair was completed with Pavemend™. A shallow depression in a concrete pavement was created with a jackhammer to a depth of 38.1 mm (1.5 in.) and 4.9 m (16 ft) by 3.0 m (10 ft) in area. The depression was filled with a 10-bucket mix of Pavemend™. A subsequent attempt was made using a large batch to repair multiple spalls; however, the material began to set in the mixer before it could be poured into the last repair.

Although relatively new to the PCC repair industry, Pavemend™ has received widespread acceptance for use as a concrete repair material for the military. These previous studies highlight the importance of following the manufacturer's instructions for mixing and placing of all rapid repair materials, as well as understanding the limitations of the material during use in a field environment.

Portland Cement Grouts

Throughout the history of airfield damage repair, various types of grouts have been used and evaluated as a material for rapid repair. A grout is defined as a mixture of binder material and water, with or without filler (UFGS, 2004).

UFC 3-270-07 (Unified Facilitates Criteria, 2002) describes a stone and grout method as a current technique for a sustainment repair for small and large craters (Figure 1). This method calls for material larger than 305 mm (12 in.) to be broken down in size and standing water to be removed from the repair hole. Protruding steel should be cut off and removed, and then the crater is backfilled with usable debris (ejecta) until the top of fill is 710 mm (28 in.) below the pavement surface. The backfill is compacted and then a 300 mm (12 in.) layer of crushed stone is placed in two lifts. A 25 to 50 mm (1 to 2 in.) layer of sand is then placed around the edges of the crushed stone to prevent seepage of the grout around the sides. An impervious membrane is then placed on the surface of the crushed stone to prevent penetration of the grout. A grout is then mixed at a 0.45 water/cement ratio and with both a friction retarder and calcium chloride. After pouring the grout into the hole, the stone is placed on top of the grout and worked in with a dozer or other piece of heavy equipment. Finally, a vibratory compactor is used to bring the grout up to the surface and provide a smooth finish. The guidance for this method states that the mix should obtain 10.3 MPa (1500 psi) compressive strength in 24 hours.

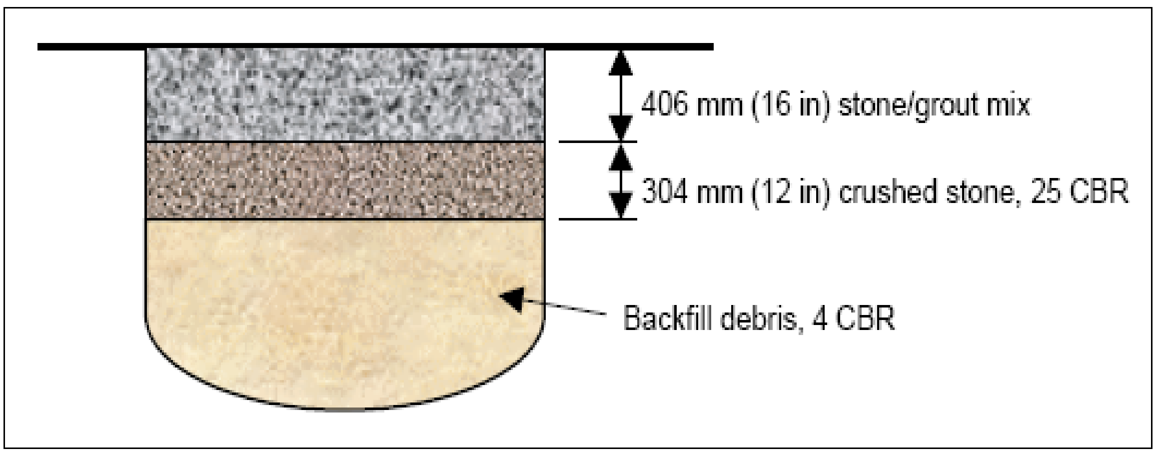

Figure 1. Cross section of stone and grout repair method from UFC 3-270-07

Stroup (1986) reports on the use of stone and grout repairs during an exercise in Europe. Various placement techniques were used and different consistencies of grout were also used. The typical grout consisted of ordinary portland cement, flake calcium chloride, cement friction reducer, and water. The density of the grout was 1,920 kg/m^3 (120 lb/ft^3), and contained percentages of 68 percent cement, 31 percent water, 1 percent calcium chloride (by weight), and 0.2 percent friction reducer by weight. These repairs were trafficked with an F-4 aircraft wheel loaded to 11,565 kg (25,500 lb) and a tire pressure of 1.97 MPa (286 psi) after 7 days with satisfactory results on two of the three craters. Stroup notes that the failure of one of the craters was due to both contamination of the stone with dust and fine aggregate, which prevented penetration of the grout, and a large amount of water in the repair hole due to a recent rain event.

Although, flowable grouts with pre-placed aggregate have been used successfully in various military applications, the method is frequently plagued with failures due to poor penetration of the grout (Hammitt, 1985, Stroup, 1986, and Thomas, 2004). A lack of penetration is typically caused by contaminated aggregate or improperly designed grout or aggregate gradation. To ensure flowability of the grout, ASTM C939 (ASTM, 2002b) can be used to measure the time of efflux of a specified volume of fluid hydraulic cement grout through a standardized flow cone. Although the method does not give results in fundamental units for rheological properties, it does provide an indication of the grout's ability to penetrate voids, such as those in pre-placed aggregate (Ferraris et al., 2001). The method is intended for efflux times of 35 seconds or less, while an alternative method involving a flow table apparatus (ASTM C230) is suggested for higher efflux times. Relative to the flow cone, the flow table is not as well suited for the JRAC scenario because the table is not easily transportable and the procedure is more complicated.

Recycled Concrete Pavement (RCP)

Recycled concrete pavement (RCP) has many uses including: aggregate for new PCC mixtures, unbound base course aggregate, cement-treated base, embankment base material, and aggregate for asphalt paving mixtures. RCP has been used successfully for all of these types of applications and, in some instances, it is a viable alternative to concrete pavement rehabilitation (CPR) or an overlay (Shelburne and Degroot, 1998).

Otsuki et al. (2003) reported that utilization of RCP is currently limited to use as a material for road subbase and backfill works due to the availability of high quality, inexpensive aggregate. They predicted an increase in the amount of concrete waste in coming years due to a shortage of disposal sites and limitations on natural resources.

Yrjanson (1989) reported that most prior uses of RCP have involved the replacement of coarse aggregate in typical PCC mixes with RCP. He states that it has not been as successful when crushed and used as fine aggregate in PCC because it is generally less workable and requires more cement due to the increased water requirements. He further states that with the equipment currently available, all types of PCC can be recycled, including reinforced concrete and even continuously reinforced pavements. At least one-third of state departments of transportation (DOTs) have used or actively use RCP material in pavement construction projects (Yrjanson, 1989).

Thirumalai (1992) reported that recycled asphalt pavement (RAP) and RCP have a substantial history of usage in pavement construction. He reports that cost savings of 20 to 50 percent can be realized by using RAP, compared to new paving materials. One state DOT study showed that the costs of using RCP would be equal to using virgin aggregates, when disposal and all other project costs were considered (Ahmed, 1991). This makes RCP more attractive in areas that do not have access to low cost, high quality aggregate.

The Corps of Engineers (Headquarters, Department of the Army, 1999) has used RCP on many projects to produce materials for stabilized base courses and new PCC pavements. The USACE recommends that when recycling D-cracked concrete, the concrete should be crushed to a maximum particle size of 19 mm (3/4 in.). Although there is little long-term experience in recycling with concrete under chemical attack (e.g., alkali-silica reaction and sulfate attack), Collins and Ciesielski (1994) reported that RCP from pavements distressed by either D-cracking or alkali-silica reaction can produce sound durable concrete for pavements. In contrast, the current DoD criteria for pavement design, UFC 3-260-02 (Unified Facilities Criteria, 2001b), states that recent problems have been encountered when using recycled concrete on a project where the material came from the local area and was used as fill and base course for a newly constructed pavement section. High levels of sulfates were present in the recycled material, which resulted in heaving of the pavement that occurred even though the recycled concrete was originally designed to be sulfate resistant and had existed in the same environment for over 30 years without issue (Rollings et al., 2006). Of course, exposure as a fill material and a base course material would be expected to be more extreme than if the material was used as aggregate in a concrete mixture and surrounded by dense mortar. Any RCP material which will be used in a concrete mixture should be evaluated with the same methods used on a virgin aggregate concrete mixture.

Otsuki et al. (2003) conducted an investigation into the interfacial transition zones (ITZ), which are the areas between the two components in concrete: the mortar matrix and the coarse aggregate particles. They report that the ITZ can significantly influence the properties of concrete and can cause concrete to fail at considerably lower stress levels than the strength of either of the two components (aggregate and mortar matrix). Mehta and Monterio (1986) similarly reported that voids and micro cracks in the ITZ do not permit stress transfer between these two components and can result in lower strengths than the individual components. Concrete made with RCP contains more ITZs than normal concrete because of the original ITZs present in the RCP. Otsuki et al. also investigated the use of the double mixing method which reserves a portion of the water to be added after the RCP. This allows the RCP to be coated with a mixture of lower w/c ratio, and results in a stronger ITZ by inhibiting crystal growth during hydration. They concluded that the quality of RCP (in terms of adhesive mortar strength) affects the strength of concrete made with RCP when the w/c ratio is low (0.25); however, it does not affect the concrete when the w/c ratio is high (0.4 and above). Additionally, it is reported that the double mixing approach results in increased strengths, chloride penetration, and carbonation resistances for concretes using RCP aggregates.

Design and Analysis for Rigid Pavement Repairs

Intermediate repairs must perform in a manner similar to the surrounding PCC concrete. Understanding the behavior of the repair and the interaction with the parent slab justifies a review of design and analysis procedures for rigid pavements.

Rollings (1981) provides a history of the U.S. Army Corps of Engineers (USACE) design and analysis procedures for rigid pavements. He reports that the USACE has used Westergaard's analytical model of a thin plate on a dense liquid foundation (Winkler foundation) to calculate the magnitude of stress. This procedure has been used to limit the tensile stress in the bottom of the slab (with the load adjacent to the slab edge) for the design of plain concrete pavements since 1946. The parameters used for stress calculations with the Westergaard model are loading, concrete modulus of elasticity (E), Poisson's ratio (μ), slab thickness (T), and modulus of subgrade reaction (k). Rollings (1981) states that values for E and μ are generally selected as 27.6×10^6 kPa (4×10^6 lbf/in.^{2}) and 0.20, respectively.

Hammons and Ioannides (1996) note the shortcomings of the Westergaard approach: the analysis is only for a single slab and therefore doesn't account for load transfer and the Winkler foundation model doesn't account for the layered nature of a pavement foundation.

Rollings (1981) reported that when aircraft loads are applied at the edge of a slab, from 0 to 50 percent of that load will be transferred to the adjacent slab across the joint. The actual load transfer across a joint depends on many factors and is not constant. The current USACE design criteria, UFC 3-260-02 (Unified Facilities Criteria, 2001b), allows a 25 percent load reduction to account for load transfer. Rollings (1981) found this reduction to be consistent with results reported during accelerated traffic tests at Lockbourne Air Force Base, Ohio, where the efficiency of different types of joints was measured (U.S. Army Engineer Ohio River Division Laboratory, 1950). Other reports

(U.S. Army Engineer Ohio River Division Laboratory, 1959 and Grau, 1979) show that the measured load transfer at various locations around the U.S. are on average 28 percent for dowelled joints while the average for keyed joints is 37 percent. UFC 3-26-02 (2001b) states that aggregate interlock can provide adequate load transfer across a joint for new pavements or during hot weather. However, load transfer is greatly reduced if joint movements due to temperature variation and loading increase and the joint begins to open. Teller and Cashell (1958) reported that dowelled joints could be constructed to obtain an initial load transfer close to the theoretical maximum (50 percent); however, repetitive loading resulted in significant decreases in load transfer across the joint. Harrison (1997) reports that environmental changes such as temperature and the type of joint significantly affect the load transfer across joints and recommended a review of the USACE design procedure, which allows for 25 percent reduction in load due to load transfer across a joint.

The repair procedure proposed in this study must be completed in a short amount of time and therefore makes the use of load transfer mechanisms unlikely because of the lengthy and difficult installation procedures associated with dowelled or keyed construction joints. TM 5-624 (Headquarters, Departments of the Army, the Navy, and the Air Force, 1995) recommends providing a thickened edge on repairs in lieu of load transfer mechanisms to compensate for increased edge stresses.

Pavement structures are commonly evaluated using devices that involve impact loads such as the Dynatest® 8081 Heavy Weight Deflectometer (HWD), which applies a single-impulse transient load at a duration of approximately 25-30 milliseconds. The height of the drop and the number of weights can be varied to produce forces ranging from 2,950 to 24,500 kg (6,500 to 54,000 lb). The force is applied through a series of rubber cushions on top of a 455-mm (17.9-in.) plate that is in contact with the surface of the pavement. The exact force of the drop is measured with a load cell, and deflections of the pavement surface due to the applied load are measured at seven locations away from the load plate using velocity transducers. The measurements are taken at 305 (12), 610 (24), 914 (36), 1219 (48), 1524 (60), and 1828 mm (72 in.) in order to obtain a deflection basin which can be used to backcalculate the moduli of the underlying layers in the pavement.

Another useful measurement from HWD data is the impulse stiffness modulus (ISM), which can be obtained by dividing the measured applied load by the deflection of the pavement directly under the load plate. The Corps of Engineers has extensively made use of ISM for identifying differences in pavement properties, and currently considers ISM values when conducting airfield pavement evaluations (Unified Facilities Criteria, 2001c). The ISM provides a quick indication of pavement stiffness, which is related to its ability to support wheel loads.

In addition to its use for evaluating continuous pavement structures, the HWD can also be used for evaluating load transfer at joints. Hammons and Ioannides (1996) describe load transfer efficiency (LTE) as a useful measurement that is defined in terms of either deflection or stress ratio between the loaded and unloaded slab. It is described as that portion of the edge stress that is carried by the adjacent unloaded slab. Hammons and Ioannides (1996) provide the following equations:

$$LTE_\delta = w_u/w_L \quad (1)$$

where:

LTE_δ = deflection load transfer efficiency

w_u = maximum edge deflection of the adjacent unloaded slab

w_L = maximum edge deflection of the loaded slab

$$LTE_\sigma = \sigma_u/\sigma_L \quad (2)$$

where:

$LTE\sigma$ = stress load transfer efficiency

σ_u = maximum bending stress in the adjacent unloaded slab

σ_L = maximum edge deflection of the loaded slab

Because the HWD can provide deflection data for each side of a joint, the deflection based LTE (Equation 1) is the parameter more commonly used by the Corps of Engineers.

CHAPTER III

MATERIALS ANALYSIS

Introduction

This chapter presents an analysis of the materials used during the investigation and explains the development of the grout and pre-placed RCP method for the repair of damaged PCC pavements. The materials investigated are limited to RCP obtained from a representative slab, Pavemend™ grout, and a Type III portland cement grout designed for comparison purposes as well as to provide another material solution for the problem. Although the Type III portland cement grout solution will not offer an extremely fast return to service (as in three hours) it does provide a simple and less expensive option when time allows (24 hours).

Analysis of Recycled Concrete Pavement (RCP)

Many factors will influence the properties of RCP, including the composition of the original concrete pavement as well as the size and type of equipment that is used for demolition. RCP produced from different pavements in different parts of the world will certainly result in a wide range of material properties. In order for this repair method to be successful, it must be able to be accomplished with relatively small, portable equipment and require very little processing prior to placing the RCP back in the repair area to serve as pre-placed aggregate.

The major objectives of this portion of the study were:

a. To gain an understanding of the resulting gradation of RCP produced by using small repair equipment on a representative concrete pavement.

b. To characterize the physical properties of the RCP and evaluate its suitability as an aggregate.

c. To determine the minimum size gradation that can be used while still allowing the grout to penetrate the full depth of the repair.

The RCP material used in the characterization study was obtained several months prior to the construction of the repairs (Photo 1). It was obtained by using the same equipment and methods for demolition as the remainder of this study. This RCP material was produced from the thicker (460 mm or 18 in.) of the available slabs. The characteristics of the slab, procedure for demolition and excavation of the RCP, and description of the equipment used are given in Chapter IV.

The material was processed to determine the particle size distribution after demolition and excavation from the repair hole. After removal, the material ranged from large chunks 150 to 225 mm (6 to 9 in.) in diameter to fine particles passing the 0.075 mm (No. 200) sieve. Initially, the material was separated into four categories using three sieve sizes (Photo 2). The largest fraction of material consisted of the large chunks that were retained on a 76.2 mm (3 in.) sieve. The second fraction passed the 76 mm (3 in.) sieve and was retained on the 25.4 mm (1 in.) sieve. The third fraction passed the 25.4 mm (1 in.) sieve and was retained on the 19.1 mm (3/4 in.) sieve. The fourth fraction was that which passed the 19.1 mm (3/4 in.) sieve. Due to the large volume of material, a representative sample of the fraction passing the 19.1 mm (3/4 in.) sieve was obtained and processed over additional sieves to provide a complete analysis of the particle size distribution. The percentages of these fractions were then applied to the total volume and the particle size distribution is shown in Table 4.

Photo 1. RCP material after demolition and excavation from the repair hole

Significant volume losses can occur when recycling concrete pavements (Yrjanson, 1989) so it was necessary to determine if the volume obtained from excavation and separated into the largest fraction (>76 mm) was sufficient to backfill the repair area prior to pouring in the grout. The goal was to obtain enough RCP of the proper gradation to completely fill the hole so as to minimize the amount of grout required and produce a homogenous repair.

Photo 2. Processing the RCP to determine gradation

Table 4. Gradation of RCP from Slab No. 1 (460 mm thick)

Sieve Size	% Passing	% Retained
76.2 mm (3 in.)	69.3	30.7
50.8 mm (2 in.)	60.4	8.9
25.4 mm (1 in.)	46.2	14.2
19.1 mm (3/4 in.)	38.1	8.1
12.7 mm (1/2 in.)	33.1	5.0
9.5 mm (3/8 in.)	30.2	2.9
4.75 mm (No. 4)	23.6	6.6
2.36 mm (No. 8)	19.3	4.3
1.18 mm (No. 16)	16.4	2.9
600 um (No. 30)	13.7	2.7
300 um (No. 50)	8.9	4.8
150 um (No. 100)	6.2	2.7
75 um (No. 200)	5.1	1.1

The objective of this exercise was also to simplify the excavation process so that a sufficient quantity of only one sieve size is needed to place back into the repair area as pre-placed aggregate. As the allowable particle size decreases, so does the void space, making it more difficult for the grout to penetrate the full depth of the repair.

Bulk unit weight of the largest fraction (>76 mm) of material was calculated to determine the volume of space it would occupy after placement back into the hole. The bulk unit weight of material greater than 76 mm was determined by weighing the approximately 0.255 m^3 (9 ft^3) of material required to fill a box. The bulk unit weight was determined to be 1097 kg/m^3 (68.5 lb/ft^3). The volume occupied by the total amount of material larger than 76 mm obtained after excavation was determined to be slightly lower than the volume of the repair hole, thus indicating that more material was required to fill the hole completely.

In order to solve this problem, the fraction of RCP retained on the 25 mm (1 in.) sieve was scalped over a 51 mm (2 in.) sieve, and the resulting coarser fraction was added and mixed with the fraction retained in the 76 mm (3 in.) sieve. Bulk unit weight for this RCP (>51 mm) was recalculated and determined to be 1152 kg/m^3 (71.9 lb/ft^3). The additional material was estimated to provide sufficient volume to completely fill the repair hole. Physical properties for the selected RCP material (>51 mm) are summarized in Table 5.

Table 5. Measured Properties of Selected RCP (> 51 mm) Particle Size

Sample #	Specific Gravity			Density kg/m^3 (lb/ft^3)			Absorption (%)
	OD[a]	SSD[b]	Apparent	OD	SSD	Apparent	
1	2.25	2.37	2.55	2241 (140)	2360 (14)	2542 (159)	5.3
2	2.28	2.37	2.51	2278 (142)	2369 (148)	2504 (156)	3.9
3	2.28	2.39	2.56	2273 (142)	2382 (149)	2553 (160)	4.8
4	2.24	2.35	2.52	2235 (140)	2347 (147)	2517 (157)	5.0
Average	2.26	2.37	2.53	2257 (141)	2361 (147)	2523 (158)	4.6

[a] Oven Dry condition was obtained by heating the aggregate to 110 ± 5ºC for sufficient time to obtain a constant mass.
[b] Saturated Surface Dry condition was obtained by submerging the aggregate in water for the prescribed period of time and then removing the free water on the surface of the aggregate.

Rapid Setting Grouts

There are many materials available in today's market that claim to have attractive properties for use as a rapid repair material. In order for a material to be considered for impregnation of pre-placed RCP, it should meet or exceed the following standards:

a. Compressive strength of 20.7 MPa (3000 psi) prior to opening the repair to traffic (3 or 24 hours).

b. Flow cone test results less than 40 seconds.

c. Easy mixing procedures and minimal components.

d. Shelf life of 2 years or more (when properly stored).

e. Consistent properties through a wide range of environmental conditions.

f. Easily transportable or easy to obtain in local markets around the world.

Pavemend™ was selected as an example of a rapid hardening grout that can achieve the 3-hour objective, and Type III portland cement grout was selected as a material that can achieve the 24-hour objective.

Before any flowable grouts are used on a larger scale, as with that of intermediate repairs, laboratory tests must quantify the physical properties of the material and determine their suitability for use with large amounts of RCP. Because many rapid setting materials generate large amounts of heat during the early stages of curing, there was additional concern that much larger volumes of material could generate enough heat to cause problems with thermal cracking. Both the laboratory and field components of this study address the potential heat generation problem. Routine testing of materials for use in this repair technique, however, was found to not require heat generation-related testing.

Pavemend™

Pavemend™ has been in use as a spall repair material by the U.S. Military for a number of years with mostly positive results. Easy mixing procedures and rapid strength gain make it a popular selection when dealing with damaged contingency airfields. The material is available from the manufacturer in 20.4-kg (45-lb) buckets, 10.2-kg (22.5-lb) buckets, and 20.4-kg (45-lb) bags. A three-year shelf life (CeraTech, 2005) accompanies the easy-to-use buckets, which makes it an attractive material for a wide range of military applications.

Pavemend™ is mixed in small quantities by using a standard drill with a paddle mixer capable of at least 300 rpm. Single batches are mixed by adding 3.8 liters (1 gal) of water to the 20.4-kg (45-lb) buckets and mixing in the original container until a target temperature of 35ºC (95ºF) is reached (Photo 3). The temperature is best monitored using a hand-held infra-red thermal gun with a digital display, as per the manufacturer's instructions. Under normal conditions (21ºC or 70ºF), this mixing process requires approximately 9 to 10 minutes.

Photo 3. Mixing a single batch of Pavemend™ in its own container with a drill and paddle mixer

Pavemend™ can also be mixed in large quantities by using a grout mixer. This is accomplished by adding multiple containers of material and water into the grout mixer and mixing until the temperature of the mixture reaches a target value of 35ºC (95ºF) as per the manufacturer's instructions. Under normal conditions (21ºC or 70ºF), the mixing process requires approximately 12 to 14 minutes. Additional information about mixing large batches and the equipment used is provided in Chapter IV.

Numerous samples of Pavemend™ were cast for the purpose of conducting laboratory tests. These samples were obtained by mixing single batches in the original container with a drill and paddle mixer. It is known that the strength gain of Pavemend™ is dependant on the rate of reaction, which is greatly affected by the mixing temperature. In order to limit the variability of strength gain with time, the temperature of the dry material and mixing water were controlled to never exceed the range of 20ºC to 22.2ºC (68ºF to 72ºF).

Thermal and Physical Properties

To gain a better understanding of the behavior of Pavemend™, several laboratory tests were conducted to determine some of the thermal and physical properties in order to predict and understand its behavior in a field environment. Tests included specific heat (Headquarters, Department of the Army, Corps of Engineers, 1973a), thermal diffusivity (Headquarters, Department of the Army, Corps of Engineers, 1973b), and hardened density and moisture content (ASTM, 1988). The results from the laboratory thermal and physical tests are summarized in Table 6.

Specific heat tests were conducted on 102 by 203 mm (4 by 8 in.) cylindrical specimens of Pavemend™ alone after 1, 3, and 7 days of curing. Two tests were conducted at each age by breaking the specimens into nominal 25.4 mm (1 in.) pieces, as

Table 6. Summary of Thermal and Physical Properties of Pavemend™

		Testing Ages[a]			Average Values
		1 Day	3 Days	7 Days	
Specific Heat	J/kg K (Btu/lb ºF)	1058 (0.253)	1071 (0.256)	1072 (0.256)	1067 (0.255)
		1083 (0.259)	1069 (0.255)	1061 (0.254)	
Average		1071 (0.256)	1070 (0.256)	1067 (0.255)	
Thermal Diffusivity	cm^2/hr (ft^2/hr)	23.9 (0.026)	23.8 (0.026)	23.7 (0.026)	23.8 (0.026)
		23.6 (0.025)	24.0 (0.026)	24.0 (0.026)	
Average		23.8 (0.026)	23.9 (0.026)	23.9 (0.026)	
Thermal Conductivity	watts/m ºC (Btu/ft hr deg ºF)	1.400 (0.866)	1.510 (0.872)	1.503 (0.869)	1.504 (0.869)
Hardened Density	kg/m^3 (lb/ft^3)	2129 (133)	2122 (133)	2129 (133)	2126 (133)
		2116.0 (132)	2128.9 (133)	2128.9 (133)	
Average		133	133	133	
Moisture Content	% of dry mass	16.9	17.3	15.1	16.4
		16.9	17.3	14.8	
Average		16.9	17.3	15.0	

[a] Test Age represents the age at which the test began. Specific heat and thermal diffusivity tests require several days to complete.

specified by the testing procedure, and then cycling the material between the temperatures of 4.4ºC and 38ºC (40ºF and 100ºF). The average specific heat was found to be 1070 J kg K (0.255 Btu/lb per degree F). The results did not vary as a function of testing age and the values are similar to the expected range for ordinary portland cement concrete of 837 to 1172 J kg K (0.20 and 0.28 Btu/lb per degree F), as reported in Neville (1995).

Thermal diffusivity tests were conducted on 102- by 203-mm (4- by 8-in.) cylindrical specimens with type T thermocouples cast at the specimen centroid. Two tests were conducted at the ages of 1, 3, and 7 days. Specimens were cycled between two constant temperature baths set at 4.4ºC and 60ºC (40ºF and 140ºF). A total of four complete cycles of hot to cold and cold to hot were performed on each specimen. Results are typical of a sanded mortar with a relatively low average diffusivity of 23.8 cm^2/hr (0.026 ft^2/hr). There was no difference in thermal diffusivity as a function of age between 1 and 7 days. Typical values of thermal diffusivity for ordinary concrete are reported by Neville (1995) as 18.6 to 55.7 cm^2/hr (0.02 to 0.06 ft^2/hr).

Hardened density tests were conducted on the specimens before their use in the specific heat tests. Average density was determined to be 2,130 kg/m^3 (133 lb/ft^3), which is similar to the typical values of portland cement concrete reported by Neville (1995) as 2,240 to 2,560 kg/m^3 (140 to 160 lb/ft^3). Average moisture content was determined to be 16.4 percent.

Unconfined Compressive Strength

Samples of Pavemend™ were cast in accordance with ASTM C 192 (ASTM, 2002a) and tested for unconfined compressive strength (ASTM C 39, 2004) to evaluate the rate of strength gain. Sample sizes were 102- by 203-mm (4- by 8-in.) cylinders. Two types of samples were cast for testing, including Pavemend™ by itself (neat) and Pavemend™ poured into molds containing a representative amount of RCP. The purpose was to evaluate the bonding properties and determine if a large amount of RCP used as aggregate would significantly affect the strength of the system. Because the RCP was used in a representative amount compared to that of a full size repair, the unconfined compressive strength was also used to provide an indication of the suitability of the RCP as an aggregate.

The RCP material used in the compressive strength testing was the fraction passing the 25-mm (1-in.) sieve and retained on the 19-mm (3/4-in.) sieve, as described early in this chapter. It was selected because it contained enough voids to allow the Pavemend™ grout to penetrate the 102- by 203-mm (4- by 8-in.) cylinder. Also, the calculated bulk unit weight of 1,110 kg/m^3 (69.5 lb/ft^3) is very similar to that of the RCP material used in the full scale testing. Both of these size fractions of RCP resulted in approximately a 1 to 1 ratio of grout to RCP by weight, making possible a strength comparison.

The RCP material used in the compressive testing was not washed and contained a reasonable amount of dust covering the outside of the material. It is not expected that engineers in the field will have the time or facilities to wash the RCP prior to use, so this condition will simulate actual "worst case" field conditions.

Three replicates of 102- by 203-mm (4- by 8-in.) samples were cast (Photo 4) for each of four test ages (3 hours, 24 hours, 7 days, and 35 days) to determine the rate of strength gain. The samples were stored in a temperature and moisture controlled facility in accordance with ASTM C 192. The average results of the compressive testing are shown in Figure 2.

Photo 4. Pouring the Pavemend™ 4-in. by 8-in. samples for unconfined compressive testing

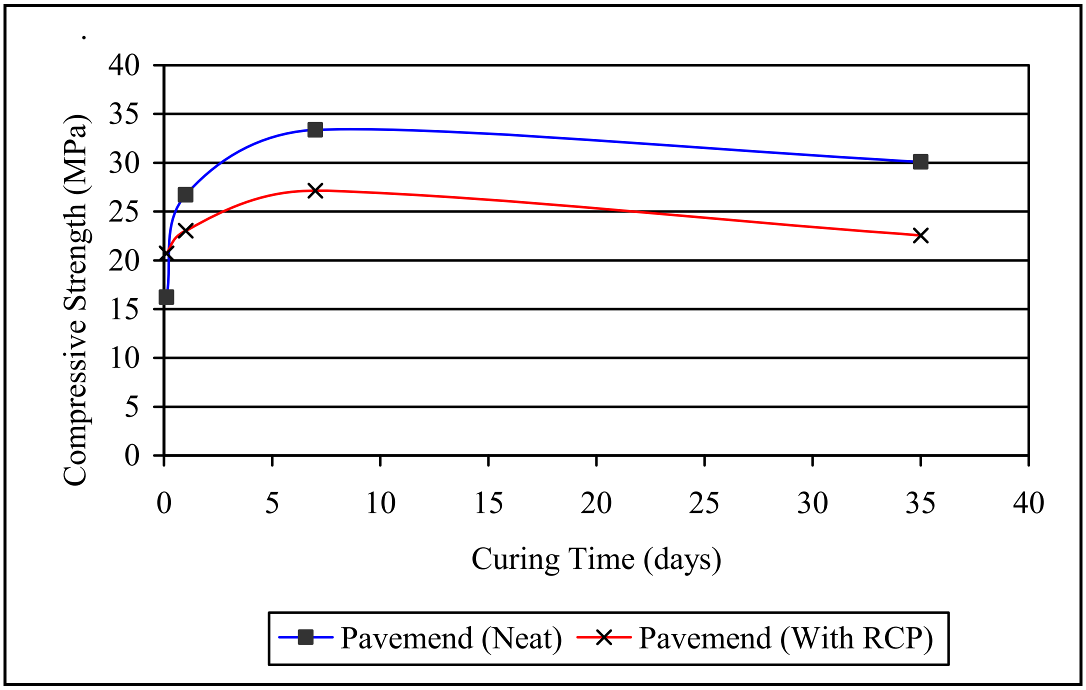

Figure 2. Compressive strength of Pavemend™ versus curing time

Overall, the results of the compressive tests indicate a slight reduction in compressive strength when using the RCP and Pavemend™ together compared to Pavemend™ as the single component. The 3-hour compressive strength test results were actually higher for the Pavemend™ and RCP combination, but a consistently lower value (approximately 20 percent lower) was obtained from the 7-day and 35-day test results. A reduction in compressive strength should be expected from the addition of 50 percent aggregate due to the voids and micro cracks which form in the interfacial transition zone (ITZ) between the two components (Pavemend™ grout and RCP). However, the compressive strength test results of the Pavemend™ and RCP system indicate that the minimum strength objective (20.7 MPa or 3000 psi) can be achieved in the required time (three hours) using this combination of materials.

The results also indicate a slight reduction in compressive strength of Pavemend™ and Pavemend™ plus RCP between the 7-day and 35-day test results. The tests were repeated using the exact same mixing, casting, and curing procedures to verify the results. The samples were tested at 35 days and again returned strengths which were lower then the compressive strength of cylinders tested at 7 days. Additional studies are needed to confirm the results and determine if the trend of decreasing strength with time continues to occur after 35 days.

Adiabatic Heat Signature Analysis

A side effect of the rapid strength gain common in many rapid setting cementitious materials is the large amount of heat that can be produced early in the curing process. It doesn't appear to cause any problems when the materials are used in small quantities such as spall repairs; however, there is concern regarding the effects of heat generation when using much larger volumes, and the implications should be understood before using the materials in the field. One method for studying these characteristics for typical portland cement concrete is the adiabatic heat signature technology developed by Quadrel™.

The equipment used for this analysis consists of a large calorimeter capable of accepting 102- by 203-mm (4- by 8-in.) samples as well as 152- by 305-mm (6- by 12-in.) samples. The system collects and logs temperature data via a probe inserted into the center of the specimen and transfers these data to a computer. The accompanying software allows for easy analysis of adiabatic temperature rise and also contains a simulation module which allows for the prediction of field performance based on properties obtained from the calorimeter measurements.

The system was designed and manufactured for the portland cement concrete industry and therefore presents some challenges when used for analyzing non-traditional materials. Ordinarily, the system is programmed to take its first reading 15 minutes after placing the sample inside the temperature-measurement chamber and then every 15 minutes thereafter until the test is stopped. This scenario is acceptable for portland cement because it produces very little heat during the first 15 minutes after mixing; therefore, the calorimeter captures the majority of the heat production during hydration of the cement. Pavemend™, however, is typically mixed until it reaches a target temperature (35ºC or 95ºF for the formula used in this study), and a significant amount of heat has already been produced when the device takes its first measurement 15 minutes after the test has begun. In one instance, the first temperature measurement was taken by the system 20 minutes after mixing Pavemend™ was completed and the material temperature was approximately 65.6ºC (150ºF). Clearly, the reaction was already well underway, thus important data were missed by the calorimeter. Due to the extremely quick reaction (peak temperature occurs approximately 4 hours after mixing) and the inconsistency with the time of the first measurements, the results were erratic and not repeatable.

Alternative mixing methods were attempted by reducing the target temperature of mixing to 26.7ºC (80ºF), thereby slowing down the rate of reaction. Clearly, this affects the speed of the reaction and therefore the behavior of Pavemend™. While not appropriate for simulating performance in the field and not ultimately useful for this study, the results are presented for completeness and for interest. The measurements obtained by such a procedure may be useful for helping to ensure quality control of the manufactured material. Also, future similar studies on rapid repair materials may benefit from the experiences shared herein. Potential solutions for this problem of temperature measurement include changing the time interval between measurements to a much lower number (e.g., 30 seconds) and somehow modifying the setup to allow the mixing to occur while the material is already in the calorimeter.

Portland Cement Grout Design

The objective of this portion of the study was to obtain a simple, reliable, and easy to reproduce mixture design that could be used as a material to meet the 24-hour objective as previously outlined. The material needed to include a standard ASTM C 150 portland cement, be fluid enough to penetrate the voids of the RCP, and gain sufficient strength in 24 hours to support the weight of large military cargo aircraft. Type III portland cement was selected as the standard material for this grout design due to its rapid strength gain properties and long history of predictable results.

The premise of the Joint Rapid Airfield Construction (JRAC) program requires that the materials used for the rapid repair scenario must be easy to transport via aircraft or be easy to obtain via the local economy throughout the world. Type III portland cement is a good choice because it is widely available throughout the world. Although the quality of cements can vary significantly from location to location, portland cements are typically manufactured to meet standard quality specifications for the market or country for which they are to be used.

The austere JRAC environment makes it difficult to obtain modern, manufactured materials locally. The difficult working conditions and potential for untrained personnel conducting the repair task suggest that the portland cement grout mixture should consist of only a few materials and be easily produced with standard equipment. For this reason, all types of admixtures were excluded from the study and the only materials considered were Type III cement, sand, and water. Although the simplest mixture design would consist of only cement and water, sand was included due to the potential positive effects such as increasing the strength and reducing the potential for shrinkage.

Five trial mixtures were originally selected to begin the study. The mixtures were batched according to the mix proportions shown in Table 7. The mixtures were then tested for flowability using the flow cone method (ASTM C 939, 2002), and 76.2-mm-(3-in.-) diameter cylinders were cast to determine the 1-day compressive strength. The flow cone apparatus is shown in Photo 5. The objective was to create and test the different mixtures to determine which one best meets the needs of the JRAC rapid repair scenario. The experiment was designed to begin with higher water/cement ratios and then decrease the ratio until the material was on the verge of not penetrating the voids of the RCP, which would achieve a balance between strength and flowability. The Pavemend™ flowability was used as the target because it had demonstrated sufficient ability to penetrate voids in pre-placed aggregate. Sand would also be introduced to the mixtures to determine if the positive effects would outweigh the burdens of an additional material.

Table 7. Trial Mixtures for Type III Grout

			Batch Weights for 2830 cc (0.1 cu ft)		
Mixture #	Name	W/C Ratio	Cement	Sand	Water
1	WC6W/OS	0.6	3078	0	1850
2	WC6WS	0.6	2186	2186	1307
3	WC5WOS	0.5	3459	0	1730
4	WC4WOS	0.4	3918	0	1584
5	WC3WOS	0.3	4596	0	1369

The sand used in Mixture 2 was typical concrete sand (ASTM C 33, 2003) purchased locally in Vicksburg, Mississippi. Laboratory tests were conducted on the material to determine properties required for the mixture design, including sieve analysis and specific gravity. The gradation is shown in Table 8 and the specific gravity was found to be 2.66.

Photo 5. Flow cone test used in ASTM C 939 procedure

Table 8. Gradation of Concrete Sand Used in Grout Design

Sieve Size	% Passing
4.75 mm (No. 4)	100.0
3.35 mm (No. 6)	100.0
2.00 mm (No. 10)	99.7
1.18 mm (No. 16)	96.3
0.85 mm (No. 20)	91.2
600 um (No. 30)	80.3
425 um (No. 40)	59.1
300 um (No. 50)	25.8
212 um (No. 70)	4.9
150 um (No. 100)	0.9
106 um (No. 140)	0.3
75 um (No. 200)	0.1

The first three mixtures were batched according to the proportions in Table 7 and the tests results are presented in Table 9. A batch of Pavemend™ was also mixed and tested for comparison purposes. After mixing the first three trial mixtures, it became apparent that reducing the water/cement ratio below 0.5 would create a mixture that would not be fluid enough to penetrate the voids of the RCP with particles greater than 51 mm. It was also observed that the addition of the fine aggregate (sand) greatly reduced the flowability of Mixture 2 without a large benefit in strength gain. Mixture 3 provided flow cone results within the required range and visibly appeared fluid enough to penetrate the voids. The rate of strength gain was also adequate with an average 1-day compressive strength of 20.57 MPa (2983 psi). The results of these tests indicated that the most promising mixture design was Mixture 3 and was therefore selected to undergo the full-scale penetration testing.

Table 9. Results of the Type III Grout Scoping Study

Mixture	Water Cement Ratio	Sand	Replicate	Compressive Strength MPa (psi)	Flow Cone (sec)
1	0.6	No	1	11.5 (1670)	18
			2	13.0 (1887)	
			3	12.9 (1873)	
			Avg	12.5 (1810)	
2	0.6	Yes	1	12.0 (1738)	40
			2	13.2 (1916)	
			3	14.0 (2023)	
			Avg	13.0 (1892)	
3	0.5	No	1	21.5 (3117)	35
			2	19.8 (2866)	
			3	20.5 (2966)	
			Avg	20.6 (2983)	
Pavemend™ 30	N/A	N/A	1	26.9 (3895)	47
			2	27.8 (4028)	
			3	26.1 (3785)	
			Avg	26.9 (3903)	

Impregnation Tests

As a stated objective, flow cone test results of 40 seconds or less provide an indication of sufficient flowability of the material; however, it does not guarantee that the material will penetrate the selected gradation of RCP aggregate. To ensure that both grouts would penetrate the full depth of a repair, boxes were constructed out of 19.1-mm (3/4-in.) plywood for the purpose of conducting impregnation tests. The inside dimensions of each box were 570 mm by 570 mm by 460 mm deep (22.5 in. by 22.5 in. by 18 in. deep). The boxes were weighed to determine the empty weight of each box (16.3 kg or 36 lb) and then filled with > 51 mm RCP material that is described earlier in this chapter. The boxes were filled to the open top and a straightedge was used to ensure that no particles extended above the surface of the box. The boxes are shown in Photo 6.

Photo 6. Plywood boxes used for impregnation tests filled with Recycled Concrete Pavement (RCP)

Box 1 contained 152.4 kg (336 lb) of RCP material and was filled with Pavemend™ as the grout material. The Pavemend™ was mixed using a portable grout mixer (Photo 7). Two batches were mixed, with each consisting of four buckets of Pavemend™ and 15.1 liters (4 gal) of water.

Photo 7. Mixing Pavemend™ material in portable grout mixer

Once the mixture reached the target temperature, it was poured into the box (Photo 8) and the mixer was immediately washed with water to eliminate material from setting inside the mixer. Once clean, the second batch was added to the mixer and completed in the same manner. Both batches started with material temperatures of approximately 21.1ºC (70ºF), and it took 13 minutes of mixing for each batch to reach the target temperature.

After pouring the majority of the second batch into the box, care was taken to achieve a smooth surface on the top of the sample. As the last of the material was leaving the mixer, the material became very thick and difficult to pour. Some imperfections can be seen on the surface of the Box 1 sample (Photo 9) due to the material setting prior to finishing the pour.

Photo 8. Pouring the Pavemend™ 30 material in the plywood boxes after mixing

Photo 9. Box 1 filled with PaveMend™ 30 and Recycled Concrete Pavement (RCP)

Box 2 contained 165 kg (364 lb) of RCP material and was filled with Type III portland cement grout. In order to ensure that there would be enough material to fill the box as well as conduct several other laboratory tests, a mix of 0.14 m^3 (5 ft^3) of material was batched. The grout was obtained by mixing 173.0 kg (381 lb) of Type III cement with 86.5 kg (191 lb) of water in a large cylindrical grout mixer that contained a pneumatic paddle type agitator located in the bottom of the mixer. The materials were weighed out on portable scales (Photo 10) and then mixed in the grout mixer for approximately 10 to 12 minutes. The material was then poured into Box 2 via a spigot in the bottom of the mixer until the box was completely filled (Photo 11). Some bleed water was observed on the surface of the Box 2 sample within several hours after the initial set of the grout.

A flow cone test (ASTM C 939) was conducted on the Type III grout and had a measured flow time of 35 seconds. Although the ASTM C 939 standard states that this test method may not be appropriate if the time of flow exceeds 35 seconds, it was used anyway due to the lack of another quick, easy, and repeatable test to measure how well a grout will penetrate voids in this type of repair. Although the grout had a faster flow time than Pavemend™ 30 (which is normally around 45 seconds), it didn't appear to penetrate the voids as easily and seemed to cling to the debris as it flowed down causing it to penetrate slower than the Pavemend™.

Photo 10. Obtaining the correct amount of water for proportioning the Type III portland cement grout

Photo 11. Pouring the Type III grout into Box II after mixing

After curing for several days, the boxes were placed under a large diameter concrete saw and were cut down the middle into two pieces for inspection (Photo 12). The grout materials successfully penetrated both box samples all the way to the bottom. The Pavemend™ material in Box 1 appeared to bond very well with the RCP in the box and no noticeable inconsistencies were observed (Photo 13).

The Type III grout in Box 2 appeared consistent throughout the box as well; however, some air voids were noticeable on the underside of several pieces of RCP aggregate (Photo 14). This is due to a combination of the slow-moving grout encapsulating the aggregate and trapping air bubbles underneath as well as bleed water which gets trapped underneath the aggregate as it rises to the surface. These air voids could be expected to weaken the bond between the grout and the aggregate; however, they appeared to be rather sparse and the impact is therefore expected to be negligible.

Photo 12. Large diameter concrete saw used to slice the box samples for inspection

Photo 13. Box 1 cut in half to permit inspection of impregnation

Photo 14. Box 2 cut in half to permit inspection of impregnation

CHAPTER IV

FIELD EXPERIMENT

Experimental Design

The field experiment for this study was designed to validate the concept of using pre-placed recycled concrete pavement (RCP) and flowable grouts to rapidly repair portland cement concrete pavements. Eight repairs were constructed on two different concrete slabs using two different materials and two levels of preparation. Demolition for Repairs 1 through 4 was initiated several months in advance of the planned construction of the repair holes in order to obtain samples of the RCP, which could be used for characterization and laboratory testing. The analysis of the RCP obtained from this excavation is presented in Chapter III.

Repairs 1 through 4 were constructed on an existing concrete slab located in the Hangar 4 Pavement Test Facility at the U.S. Army Engineer Research and Development Center (ERDC) in Vicksburg, Mississippi. The dimensions of the slab were 7.3 m long by 12.2 m wide by 460 mm thick (24 ft long by 40 ft wide by 18 in. thick) and it was placed directly over a sand base layer. The test slab was originally constructed in the spring of 1990 for a project in which fiberglass-reinforced plastic (FRP) panels were being evaluated for F-4 and C-141 aircraft traffic (Grau and McCaffrey, 1990). The slab was one of two that were used to provide anchoring for the FRP panels so that traffic could be applied in order to evaluate the performance of the mat systems under actual aircraft loading. This particular slab had a 1.8 m (6 ft) wide section of asphalt overlay running along one length of the slab in order to simulate such a pavement in the FRP experiment. Only Repair 1 was located in this region of the slab. The asphalt surface was even with the top of the adjacent slab and was approximately 102 mm (4 in.) in depth. The asphalt was placed on top of the concrete as in a flexible overlay for a rigid pavement. The layout of Repairs 1 through 4 is shown in F.

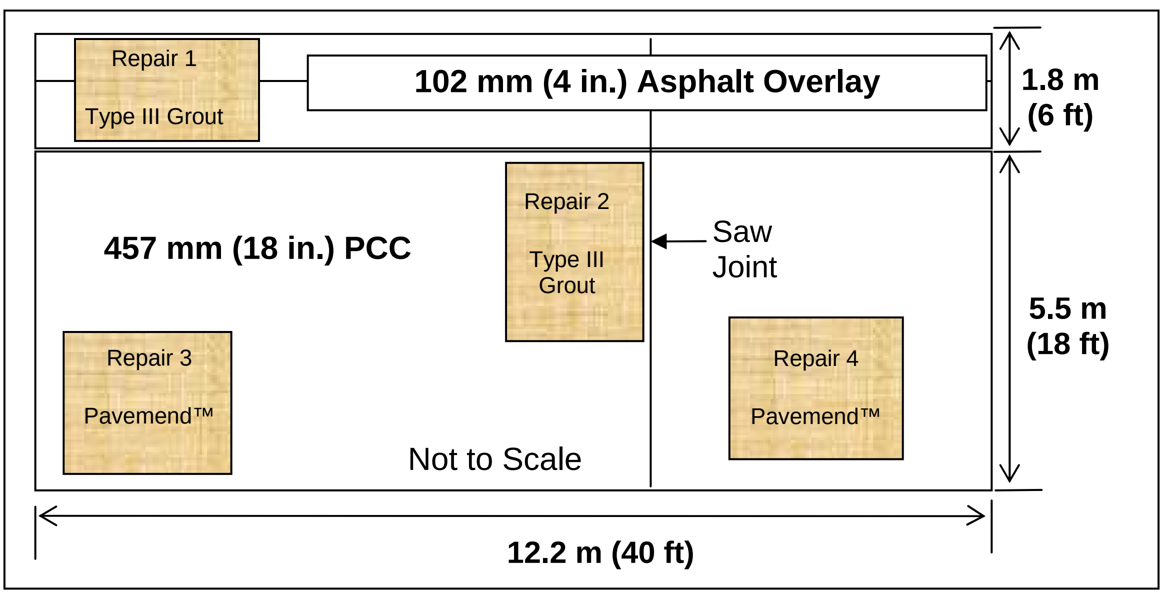

Figure 3. Location of Repairs 1 through 4

Repairs 5 through 8 were constructed on an existing slab on the opposite end of the ERDC Hangar 4 facility. The dimensions of the concrete were 4.3 by 18.3 m (14 by 60 ft) and the slab thickness was 240 mm (9.5 in.). The slab was placed over a

510-mm- (20-in.-) thick crushed limestone base course and the 90-day compressive strength was 37 MPa (5400 psi). It was constructed in January of 2004 for the purpose of testing rapid setting materials for spall repair. The slab contained numerous areas where the repair tests were conducted, however there was also sufficient room to locate Repairs 5 through 8 of this study on undisturbed concrete. The layout of Repairs 5 through 8 is shown in Figure 4.

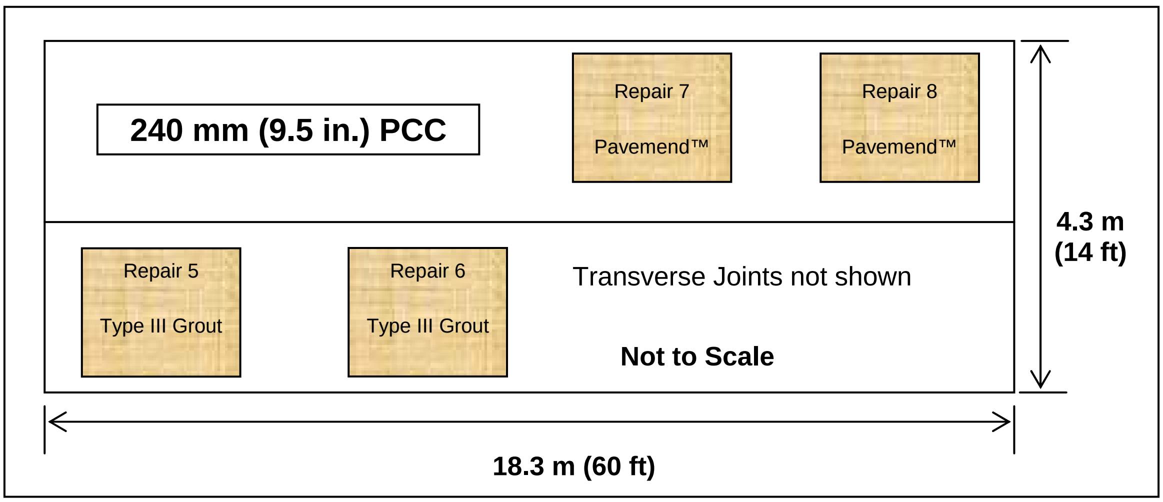

Figure 4. Location of Repairs 5 through 8

The two existing slabs discussed above provided a unique opportunity for this study for several reasons. First, they eliminated the need for the costly construction of new slabs to conduct the testing. Secondly, they provided a good range of characteristics which might be encountered in the field including slab thickness, age of the concrete, different types of aggregate used in the concrete, and also different materials directly under the slab (sand and crushed limestone).

Rapid Repair Construction Procedures

The construction procedures implemented in this study needed to take into account the potential lack of available time and equipment during any rapid repair operation. It should be noted that there are many additional steps that can be taken to increase the quality of repairs in situations where time and equipment are readily available. However, for this study, it was assumed that time and resources would not allow many of these steps and it therefore represents a "worst case" scenario for the construction procedures. If time and equipment are available, then every effort should be made to take the extra steps which are discussed in the following section (e.g., removal and replacement of the base course with a high-quality crushed aggregate).

Removal of Deteriorated Pavement

The first step in the rapid repair process is to identify the area of failed pavement and mark the area to be repaired. ETL 02-19 (Headquarters, Air Force Civil Engineering Support Agency, 2002) describes the procedures used to evaluate pavements in a contingency environment and should be used to determine the areas in need of repair. The marked area should extend slightly beyond the visibly distressed area to ensure that the repair will be in contact with sound concrete. It is helpful to mark the outline of the repair with a fluorescent paint so that the repair is visible to operators during the sawing operations (Photo 15). The repairs in this study were excavated using a Bobcat™ skid steer type loader with several attachments.

Photo 15. The outline of the repair marked with fluorescent paint

First, a concrete wheel saw attachment was used to cut the outline of the repair. Once all four sides of the repair were cut to the appropriate depth the pneumatic pavement breaker attachment was used to break up the deteriorated concrete so that it could be removed. The larger slab used for Repairs 1 through 4 required significantly more effort to break up the interior of the repair area compared to the smaller slab used for Repairs 5 through 8 due to harder aggregate (chert vs. limestone) and the older age of the larger slab. During the concrete breaking process for the larger slab, the loose material on top had to be removed by hand several times during the process to allow the operator to continue breaking the slab down to its full depth (Photo 16). The smaller slab was much easier to break because the pneumatic pavement breaker would easily penetrate the full depth of the slab, which produced much more uniform pieces of RCP that were approximately 152 to 229 mm (6 to 9 in.) in diameter (Photo 17).

Photo 16. Pavement breaker attachment breaking the interior of the repair

Once removed, the loose material was placed next to the repair. Photo 18 shows one of the repairs in the larger slab after the removal of the deteriorated section and Photo 19 shows the pile of material removed from the interior of the repair. The larger slab produced a large amount of material which was smaller than 51 mm (2 in.) and therefore required processing before the material could be placed back into the repair. In contrast, the smaller slab only produced a minimal amount of material smaller than 51 mm (2 in.), and therefore did not need to be processed. The material was simply placed back in the hole once it was properly prepared.

Photo 17. RCP excavated from the smaller slab for Repairs 5 through 8

Photo 18. Repair hole after removal of the deteriorated section

Photo 19. The RCP material removed from Repair 2

If the material being removed from the repair contained material smaller then 51 mm (2 in.) in diameter, then it was screened to remove the unwanted smaller fraction before it was placed back in the hole. This ensured a sufficiently porous gradation that allows the grout materials to penetrate the full depth of the repair. This screening process was accomplished as the material was removed so that it was ready to be placed immediately back in the hole and required only single handling. For this study the screen was placed over a large container and the material smaller than 51 mm (2 in.) was allowed to fall into the container for easy disposal once the repair was complete (Photo 20). The usable portion of the RCP was then placed next to the repair and was ready for use. Although not done in this study, if a water source is available and the material is contaminated with soil or dust as a result of the demolition process, it should be washed to remove the unwanted particles. This will provide a better bond between the grout and RCP and result in a stronger and more durable repair.

Photo 20. Unusable material screened into large container (black drum)

As the demolition process reaches the bottom of the slab, care must be taken to minimize disruption to the base course and underlying layers. Some disturbance is inevitable however, and should be repaired prior to filling the hole with RCP. The base course material should be inspected and, if time allows, replaced with a high-quality crushed aggregate and compacted with a motorized hand tamper.

For this study, two levels of preparation were tested for each pair of repairs. TM 5-624 (Headquarters, Departments of the Army, Navy, and Air Force, 1995) suggests that when making repairs near joints or on slab edges, the repair should be placed approximately 51 mm (2 in.) thicker and extend approximately 51 mm (2 in.) beyond the existing slab. This practice provides a thickened edge for the repair and will reduce tensile stresses which could lead to cracking. For repairs conducted at a working joint, bond breaking systems such as polyethylene sheeting or grease should be used so that the repair does not impede the movement of the joint.

The process of undercutting the existing concrete was attempted with hand tools and was very difficult to achieve due to the small working area and the tendency of the material to roll back into the undercut area. A decision was made to simply install a 102-mm (4-in.) trench around the outside of the repair which would provide a thickened edge on the outside of the repair slab. Photo 21 shows this thickened edge technique for Repair 8. Information relating to the performance of the repairs as a result of the level of preparation is discussed later in this chapter.

Photo 21. Repair 8 with thickened edge prior to RCP placement

Placement of the RCP

Once the hole was prepared and the RCP screened (if required), the RCP was placed back in the hole (Photo 22 and Photo 23). The larger pieces of RCP were placed back into the hole first and the smaller pieces were placed closer to the top of the repair. This allows for larger void space at the bottom where the grout penetration is more critical and the smaller more maneuverable pieces at the top will make for easier finishing. Once the hole was filled with RCP, a straight edge was used to ensure that there were no pieces of RCP protruding above the existing surface of the concrete (Photo 24). The grout materials had a tendency to “float” some of the pieces close to the top of the repair which sometimes resulted in an undesirable rough surface. If this occurred, then the pieces were simply removed during grout placement and the surface void was filled with grout.

Photo 22. Repairs 1 through 4 with the RCP for Repairs 1 and 2 ready for placement

Photo 23. Placement of the RCP into the repair

Photo 24. Checking the surface of the repair area to ensure there is no protruding material

Grout Mixing and Placement

Mixing of the grout was accomplished using a grout mixer. The type of mixer used in this study is explained in more detail later in this chapter. A grout mixer works by rotating paddles (usually lined with rubber) inside the mixing chamber until the material has been properly mixed, and then the contents of the mixer are poured directly into the repair hole. The advantage of a mortar mixer, versus a concrete mixer, is that the rubber-lined blades clean off the inside of the drum during mixing. The mixing also imposes more shearing action to the grout. The two materials used in this study had different mixing procedures and therefore are described separately.

Type III Portland Cement Grout

The design for the grout used for these repairs is described in Chapter III. The mixture consisted of Type III portland cement mixed at a 0.5 water/cement ratio. The mixing water was calculated to equal 21 liters (5.7 gal) per 42.6 kg (94 lb) sack of cement, which was measured using 19 liter (5 gal) buckets. The first measurement was made by weighing the water on scales and marking the appropriate fill locations on the buckets to ensure the correct amount of mixing water. The first two repairs using this material were made by multiple batches of two bags of cement and 43 liters (11.3 gal) of water; however, the batching was increased to three 42.6 kg (94 lb) sacks of cement and 64 liters (17.0 gal) of water once it was determined that the mixer had sufficient capacity. The estimated and actual quantities of materials used in each repair are summarized in Table 10. The mixer was washed out after several batches when a build up of material was detected on the paddles of the mixer. It was later discovered that this build-up could be minimized by always adding 19 liters (5 gal) of the mixing water immediately after pouring the previous batch and letting it wash around in the mixer for 1 to 2 minutes before adding the dry materials for the next batch.

The grout batches were mixed and poured into the hole until the surface of the repair was even with the surface of the surrounding concrete. Photo 25 shows the Type III portland cement grout being poured into the repair. The consistency of the grout was such that it was almost self-leveling; however, some of the grout near the top required finishing to achieve the desired level surface. A screed board was used to strike off the surface and normal concrete tools were used to smooth the surface when necessary. For this study, only minimal finishing was required and it was accomplished using a trowel just after the initial set of the grout.

Table 10. Summary of Repairs

Repair	Material	Volume of Repair (yd^3)	Estimated Quantity	Actual Quantity	Type of Equipment	Thickened Edge	Type of Curing
1	Type III Grout	1.19	14.1 sacks	14 sacks	Small	No	None
2	Type III Grout	0.99	11.8 sacks	12 sacks	Small	Yes	Moist
3	Pavemend	1.23	49.2 buckets	47 buckets	Large	No	None
4	Pavemend	1.02	40.9 buckets	38 buckets	Large	Yes	None
5	Type III Grout	0.42	5.0 sacks	5.5 sacks	Small	No	None
6	Type III Grout	0.37	4.4 sacks	5.5 sacks	Small	Yes	Moist
7	Pavemend	0.39	15.7 buckets	17 buckets	Small	No	None
8	Pavemend	0.42	16.9 buckets	17 buckets	Small	Yes	None

Photo 25. Pouring the Type III grout after mixing

Due to the relatively high water/cement ratio (0.5) required to provide enough workability for the grout to penetrate the RCP, there was a high likelihood for shrinkage cracking to occur on the surface of the repair, especially if no attempt was made to moist cure the repairs made with Type III grout. For this study, Repairs 2 and 6 were moist cured using soaked burlap placed on the repair after the surface hardened, approximately 4 hours after placement (Photo 26).

Photo 26. Moist curing of Type III grout using soaked burlap

Pavemend™

As described in Chapter III, Pavemend™ can be mixed in large quantities using a grout mixer. The material is packaged in 19-liter (5-gal) buckets and shipped on pallets containing 36 buckets. The product is very sensitive to temperature and fluctuations of initial material temperature and mixing water temperature can produce drastically different results in material set times. During the field placements, the initial material temperature and ambient temperature remained somewhat constant around 21.1ºC (70ºF), which is considered optimal for this material. It is highly recommended that the manufacturer's instructions be followed for temperature considerations and the use of this material should be thoroughly investigated if extreme temperatures are expected during the repair process.

The mixing water and Pavemend™ material were added to the mixer and the temperature was monitored using a hand held temperature gun as shown in Photo 27. Once the temperature of the material reached 35ºC (95ºF), the contents of the mixer were poured into the repair as shown in Photo 28. Batches were made by using 10 buckets of Pavemend™ and 37.9 liters (10 gal) of mixing water. Initially, the mixer was rinsed with water after each mix because of concerns that the material would rapidly set in the mixer. It was later discovered that this could be avoided by always adding 18.9 liters (5 gal) of the mixing water immediately after pouring the previous batch and letting it wash around in the mixer for 1 to 2 minutes before adding the dry materials for the next batch. This procedure kept the inside of the mixer relatively clean and eliminated the need to rinse it out after each batch.

Photo 27. Using the hand-held temperature gun to monitor temperature rise in the Pavemend™ material

Photo 28. Pouring the Pavemend™ material into the repair

The batches were made and the material was poured into the hole until it was level with the surrounding concrete. Unlike the Type III grout, the Pavemend™ material is completely self-leveling and does not need to be finished. Care must be taken when pouring the last lift of the material to ensure that it does not set too fast. Whenever the material is being placed on a layer that is already at elevated temperatures, it tends to speed the setting process thus causing it to lose its self-leveling properties. This can result in an undesirable and uneven finish. During this study, the mixing time of the final lift was slightly reduced and the temperature was only allowed to reach approximately 33.3 to 33.9ºC (92 to 93ºF), which provided enough working time for the material to flow and provided a smooth finish. Photo 29 shows a repair just prior to receiving the final lift of material.

Photo 29. Pavemend™ repair just prior to pouring the final lift

Equipment

Two models of skid steer type loaders were used for the excavation of the repairs. A large Bobcat™ model T300 with rubber tracks was used on two of the repairs (Repairs 3 and 4) with a large 458-mm (18-in.) wheel saw (Bobcat™ model WS18) as well as a 650 J (500-ft-lb) pneumatic pavement breaker (Bobcat™ model HB950). This relatively large-sized equipment was obtained because it was originally estimated to be necessary in order to excavate the repairs in the large 458-mm (18-in.) slab (Repairs 1 through 4).

The larger 458-mm (18-in.) wheel saw was sufficient to saw through the full depth of the 458-mm (18-in.) slab; however, there was some difficulty operating the saw when attempting to make a full 458-mm (18-in.) cut. The rotating portion of the wheel saw tended to hang up frequently, forcing the operator to back up and start the cut again. The large wheel saw was approximately 152-mm (6-in.) in width and created a substantial width of cut (178-mm or 7-in.) that generated a significant amount of dust and unusable material. After several cuts were made with the large wheel saw, it was deemed excessive for this type of repair.

A smaller-sized machine with smaller attachments was also used to determine the optimum approach for excavating the repairs. The smaller machine was a Bobcat™ 863 wheel-type loader. It was used with a 305-mm (12-in.) wheel saw (Bobcat™ model WS12) and 406 J (300-ft-lb) pneumatic pavement breaker (Bobcat™ model HB 880). The large machine and attachments were used first to excavate repair holes 3 and 4 and then the smaller equipment was used on repair holes 1 and 2 as well as repairs 5 through 8 on the smaller slab. The smaller machine and attachments were just as effective and the width of the cut seemed more reasonable at approximately 102 mm (4 in.). Although the smaller saw was not capable of cutting the entire depth of the slab, it was later determined that this was not a problem as the pneumatic pavement breaker could finish cutting the repair away from the parent slab. Photo 30 and Photo 31 show repair holes cut with the large 458-mm (18-in.) saw and the smaller 305-mm (12-in.) saw, respectively. Table 11 contains detailed information about the time required to make each repair.

The grout mixer used during this study was a Whiteman™ 0.17 m^3 (6 ft^3) capacity model with an 8-horsepower gasoline engine. There are numerous other types and sizes available on the commercial market and any of these models would be adequate for the mixing process. It is recommended by the manufacturer of the Pavemend™ material (CeraTech, 2005) that only grout mixers, and not concrete mixers, be used for the mixing process. This is due to the additional shearing effects produced by a grout mixer that rotates paddles within the drum.

Photo 30. Repair 3 after being cut with the large 458-mm (18-in.) wheel saw

Photo 31. Repair 1 after being cut with the small 305-mm (12-in.) wheel saw

Table 11. Summary of Time Required for Each Repair

Repair	Time for Excavation (min)	Time for Placement (min)	Total Repair Time (hr)	Unit Cost for Repair Materials	Total Cost for Repair Materials
1	135	90	3.75	$7	$98
2	120	45	2.75	$7	$84
3	140	120	4.33	$50	$2350
4	125	60	3.08	$50	$1900
5	90	45	2.25	$7	$42
6	80	25	1.75	$7	$42
7	75	25	1.67	$50	$850
8	75	30	1.75	$50	$850

Dynamic Cone Penetrometer (DCP) Measurements

After the repairs were excavated and all the material had been removed, DCP tests were conducted in the bottom of the repair holes to obtain information regarding the strength of the materials beneath the slab. The DCP measurements were conducted in accordance with the procedures described in Webster, Grau, and Williams (1992). The DCP is a handheld device that uses a hammer to drive a cone into the ground at the location to be tested. The DCP used in this study had a 60-degree cone tip with a base diameter of 20 mm (0.79 in.). The hammer was dropped from 575 mm (22.6 in.) and weighed 8.0 kg (17.6 lb). The measurements of the cone’s penetration and the corresponding number of hammer blows were recorded approximately every 25 mm (1 in.) or whenever a noticeable change in penetration rate occurred. This information was then used to compute a DCP index in terms of penetration per blow for the measurement intervals. The DCP index was then converted to a CBR percentage using the following correlation[1]:

$$CBR = 292/DCP^{1.12} \tag{3}$$

where DCP is in mm/blow. The above correlation has been used to create a Microsoft™ EXCEL spreadsheet which automatically computes and displays CBR with depth information. The DCP data obtained in this study were processed using the spreadsheet, and Figure 5 and Figure 6 illustrate the results of two of the DCP tests as computed by the spreadsheet. Table 12 provides a summary of all the DCP tests conducted and a calculated average CBR for the various layers selected.

A total of 14 DCP tests were conducted in Repairs 1 through 4. The material under the larger slab (Repairs 1 through 4) was visibly identified as typical concrete sand and it was estimated to be approximately 152 mm (6 in.) thick. It was difficult to determine the type of material under the sand layer; and due to prior construction activity in that area, the underlying layers are likely combinations of many different materials. The DCP tests conducted on Repair holes 1 through 4 indicated a change in strength below 6 in., which would agree with the previous estimate of base layer thickness. DCP tests conducted on Repair 1 indicated relatively higher strength values compared to the other three repair holes. This is likely due to Repair 1 being located on the inside of the

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[1] The Engineer Research and Development Center (Webster, Grau, and Williams, 1992) developed this correlation for a wide range of granular and cohesive soils and the correlation is widely accepted in the industry today (Phillips, 2005).

DCP TEST DATA

File Name: DCP2002

Project: *JRAC Rapid Repair* **Date:** 10-Mar-05

Location: Repair Hole 1 - Test 1 **Soil Type(s):** Unknown

Hammer
◯ 10.1 lbs.
◯ 17.6 lbs.
◉ Both hammers used

Soil Type
◯ CH
◯ CL
◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	20	1
1	60	1
2	80	1
2	100	1
2	110	1
2	130	1
3	150	1
3	180	1
3	220	1
1	230	1
1	250	1
1	260	1
1	270	1
2	290	1
2	320	1
2	330	1
2	350	1
2	360	1
3	380	1
3	410	1
3	430	1
3	460	1
3	490	1
3	520	1
3	540	1
3	560	1
3	590	1
3	610	1
3	640	1
3	680	1

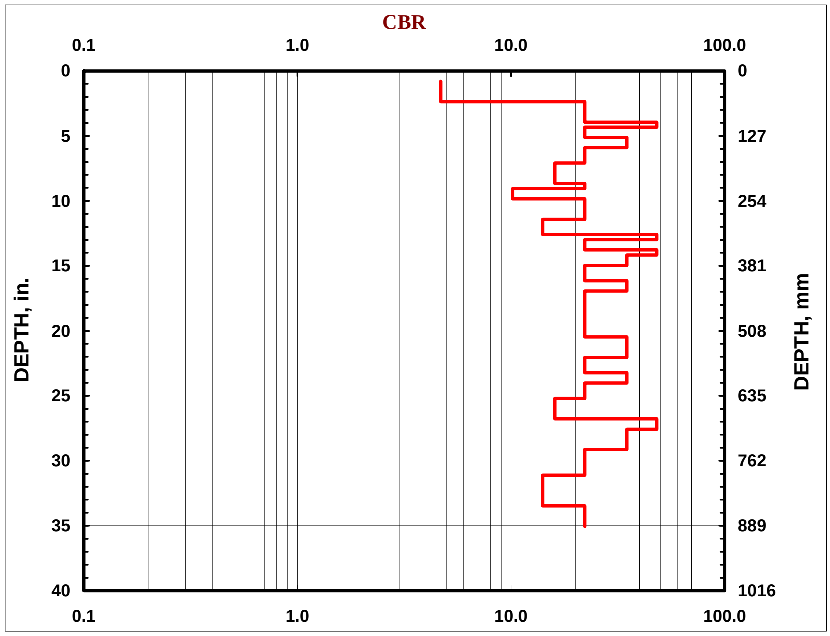

Figure 5. EXCEL spreadsheet to compute CBR using DCP measurements for Test No. 1 in Repair 1

DCP TEST DATA

File Name: DCP2002

Project: *JRAC Rapid Repair* | **Date:** 10-Mar-05

Location: Repair Hole # 3 - Test 4 | **Soil Type(s):** Unknown

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	10	1
1	40	1
1	60	1
2	95	1
2	130	1
2	160	1
2	230	1
1	270	1
1	310	1
1	330	1
1	340	1
2	370	1
2	410	1
1	440	1
1	460	1
1	490	1
1	520	1
1	540	1
1	570	1
1	605	1
1	650	1
1	710	1
1	760	1
1	800	1
1	830	1
1	870	1
1	920	1

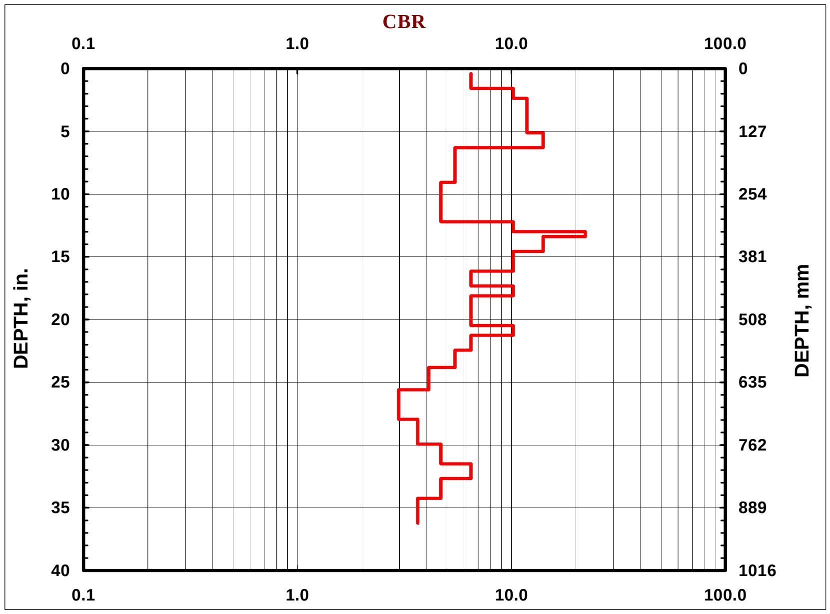

Figure 6. EXCEL spreadsheet to compute CBR using DCP measurements for Test No. 4 in Repair 3

Table 12. Summary of DCP Measurements for Repair 1 through 8[a]

Repair Hole #	DCP Test #	Avg. CBR 0 to 152 mm (0 to 6 in.)	Avg. CBR 152 to 305 mm (6 to 12 in.)	Avg. CBR (305 to 610 mm) (12 to 24 in.)	Avg. CBR 610 to 915 mm (24 to 36 in.)
1	1	30	20	30	20
	2	8	5	20	30
	3	30	15	25	25
	4	20	6	20	15
Average CBR		22	12	24	23
2	1	20	12	5	3
	2	20	15	5	6
	3	20	7	4	3
Average CBR		20	11	5	3
3	1	10	6	5	4
	2	15	10	5	3
	3	12	18	6	7
	4	11	4	8	4
Average CBR		12	10	6	5
4	1	7	20	10	3
	2	7	20	10	5
	3	20	20	10	5
Average CBR		11	20	10	4
5	1	100	100	--	--
6	1	100	100	--	--

[a]DCP measurements were converted to CBR values using procedures described in Webster, Grau, and Williams (1992).

slab in the transition zone of where the higher quality materials would have been placed during the previous study for which the slabs were constructed. Average CBR values for the underlying layers in Repair 1 ranged from 12 to 24 percent with a slightly weaker layer indicated from 152 to 305 mm (6 to 12 in.) in depth. CBR values in Repairs 2 through 4 were generally consistent with depth, and the values ranged from 3 percent to 20 percent. The higher CBR values were located in the higher layers, while the strength tended to decrease with depth and generally was in the single digits below 305 mm (12 in.).

Two DCP tests were conducted for Repairs 5 through 8. The slab used for Repairs 5 through 8 was placed on a 510 mm (20 in.) thick layer of high quality crushed aggregate base course material. Both DCP tests were terminated at approximately 152 mm (6 in.) when the cone penetration was minimal after numerous drops of the hammer. The CBR values are reported as 100 percent due to refusal during the DCP tests. The high quality material was known to be consistent throughout the area under the slab and therefore additional tests were not required.

Strength Measurements

Samples of Pavemend™ and Type III portland cement grout were cast from the large batches used to make the repairs. The samples were tested at similar ages to those in the laboratory portion of the study to determine if there is was appreciable difference in strength gain as a result of the different mixing processes. The samples consisted of only the grout material and were tested at 1, 7, and 28 days. The results are shown in Figure 7.

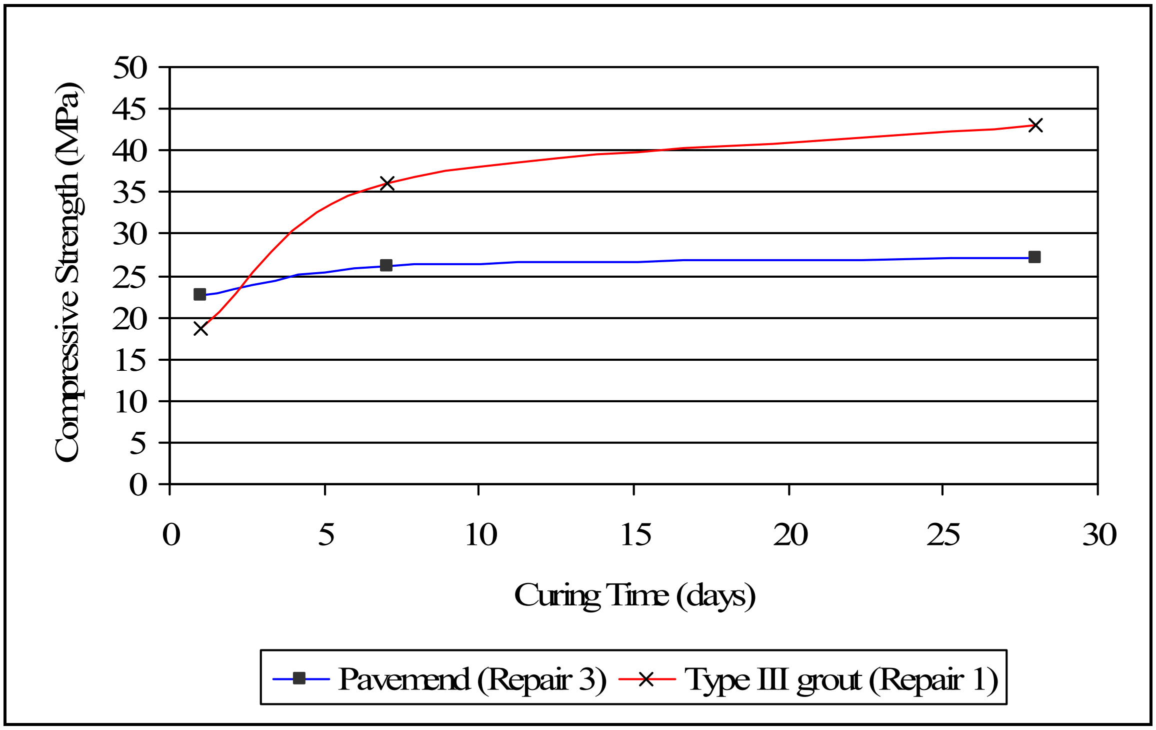

Figure 7. Compressive strength of samples cast from large batches during field placements

Figure 8 shows a comparison of the effects of the laboratory mixing procedure (drill and paddle mixer) and the field mixing procedure (mortar mixer) on the strength gain of Pavemend™. The strength development of Pavemend™ is known to be a function of the mixing temperatures of the materials, which directly affects the rate of reaction. The field mixing procedure produces less shearing compared to the laboratory mixing procedure and therefore is expected to result in a reduced rate of reaction for the material. The magnitude of difference in these mixing procedures can be observed in Figure 8 and has a significant effect on the strength gain of Pavemend™.

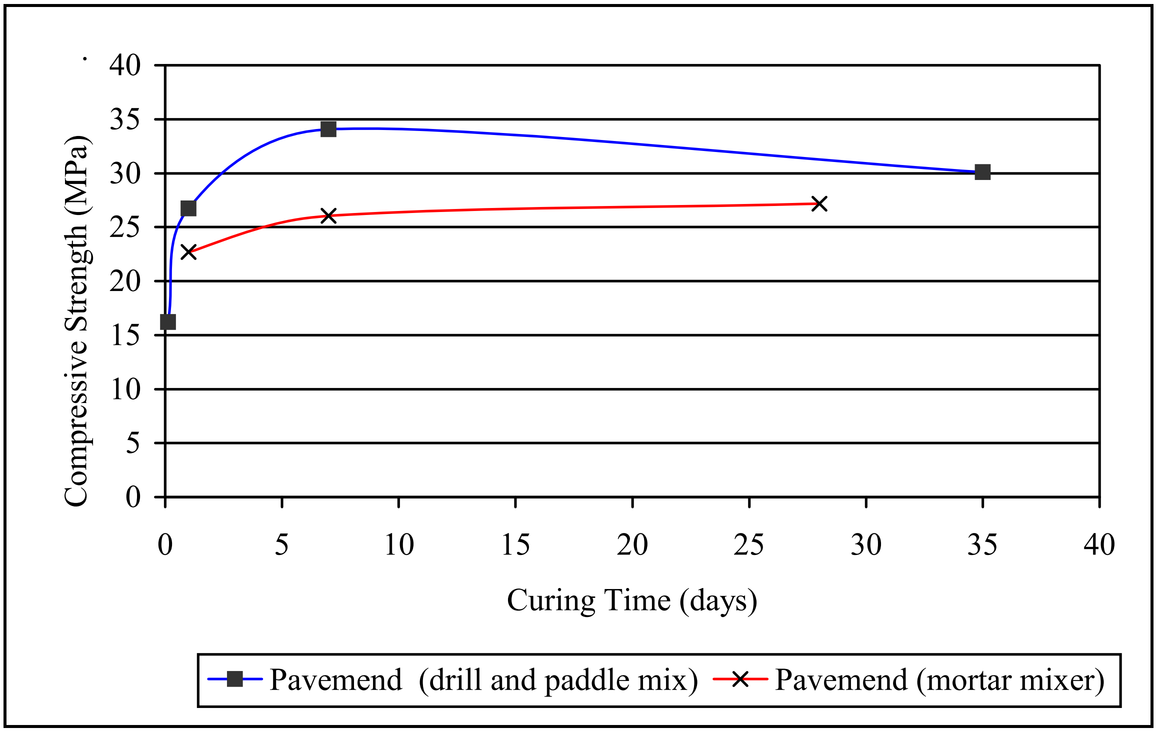

Figure 8. Effects of different mixing techniques on strength gain of Pavemend™

The 1-day strength for Pavemend™ mixed in the mortar mixer was less than the strength from the laboratory mixing procedure and the 3-hour strengths would also be expected to be less than those obtained in the laboratory. However, as will be discussed later in this chapter, all repairs made with Pavemend™ mixed in a mortar mixer successfully withstood the application of traffic after three hours of curing, implying that the strengths were sufficient. Additionally, the samples of Pavemend™ mixed in the mortar mixer did not exhibit a loss in strength between 7 and 28 days as in the laboratory study, although the gain in strength was not significant. In contrast, the Type III grout continued to gain significant strength with time and provided significantly more "reserve" strength after 1 day (Figure 7). Compressive strength results for the field castings of the Type III grout were very similar to those obtained during the laboratory portion of the investigation.

Temperature Measurements

A number of sensors were used to measure and record temperature data at regular intervals during the curing process of the materials in order to obtain information relating to the amount of heat generation in each of the repairs. The sensors were programmed to record measurements at a variety of time intervals using a simple software program. This particular sensor, manufactured by The Transtec Group, is typically used for maturity measurements in portland cement concrete because of the ease in which data can be obtained and the durability of the sensors. The individual sensors can be connected to a computer via two small electrical leads and a USB connection, which is programmed to record data at intervals determined by the user. The sensors contain internal memory which allows for approximately 2050 readings of time and temperature. Once the sensors are instructed to start recording, the lead wires for the sensors can be reattached to the computer at any time and the recorded data are instantly downloaded via the software installed on the computer. A laptop computer was used during this study and provided a very efficient means to program the sensors just prior to use in the repairs as well as download data once testing was complete. All of the sensors in this study were programmed to record data every 5 minutes for durations of just over 7 days. The sensors are shown in Photo 32. For scale, the button shaped sensors in Photo 32 are approximately 20 mm in diameter.

Photo 32. Sensors used to measure and record temperature in the repairs

The temperature sensors were used to record data in four of the eight repairs. Twelve sensors each were used in Repairs 2 and 4 and eight sensors each were used in Repairs 6 and 8. They were positioned in the repairs as the RCP was being placed and although they appeared to be durable, care was taken so they would not be damaged. The sensors were placed in three layer positions (top, middle, and bottom) in each of the repairs and at different locations (center and edges). This arrangement was selected to obtain maximum information about the temperature conditions at various locations within the repairs. The sensors were labeled and their position in the repair was recorded. The location of the sensors is summarized in Table 13. Two of the forty sensors did not provide data after being connected back to the computer. One of the failed sensors was in the bottom of Repair 6 and the other was in the middle of Repair 8. The data

Table 13. Location of Temperature Sensors for Repairs 2, 4, 6, and 8

Repair No. 2				Repair No. 4			
Sensor No.	Location	Depth		Sensor No.	Location	Depth	
		in.	mm			in.	mm
1	center	18	457	1	center	18	457
2	center	18	457	2	center	18	457
3	side edge	18	457	3	front edge	18	457
4	front edge	18	457	4	side edge	18	457
5	center	9	229	5	center	9	229
6	center	9	229	6	center	9	229
7	side edge	9	229	7	side edge	9	229
8	front edge	9	229	8	front edge	9	229
9	center	1	25	9	center	1	25
10	center	1	25	10	center	1	25
11	side edge	1	25	11	side edge	1	25
12	front edge	1	25	12	front edge	1	25
Repair No. 6				Repair No. 8			
Sensor No.	Location	Depth		Sensor No.	Location	Depth	
		in.	mm			in.	mm
1[a]	center	9	229	1	center	9	229
2	side edge	9	229	2	side edge	9	229
3	center	4.5	114	3	center	4.5	114
4	side edge	4.5	114	4[a]	side edge	4.5	114
5	front edge	4.5	114	5	front edge	4.5	114
6	center	4.5	114	6	center	4.5	114
7	center	1	25	7	side edge	1	25
8	side edge	1	25	8	front edge	1	25

*Data not retrieved due to sensor damage.

transmission error was likely due to damage to the sensor or the connecting wires suffered during the placement of the RCP.

The results of the temperature measurements are shown in Figure 9 through Figure 13. These figures show the temperature history for selected sensors installed in each repair plus the maximum temperature differential which occurred between the sensor reporting the highest and lowest reading in each of the repairs. The temperature differential was calculated because it provides an indication of the potential for thermal cracking within the repair.

Repairs 4 and 6 obtained the highest temperature measurements out of all the repairs. Although a large volume of Pavemend™ was used in Repair 4, the maximum temperatures achieved weren't significantly different than those achieved during the laboratory study using single batches. Both Pavemend™ and Type III grout achieved similar maximum values indicating that Pavemend™ doesn't get hotter than the Type III grout as was expected. The Pavemend™ repairs achieved the maximum temperatures earlier (3 to 5 hours) than the Type III grout repairs (8 to 15 hours), and cooled off quicker, as would be expected due to the rapid nature of the Pavemend™ reaction. A noticeable difference was observed between the Pavemend™ repair in the thinner slab (Repair 4), which cooled much quicker than the thicker repair (Repair 8). The Pavemend™ temperatures were higher in the larger repair compared to the smaller repair; however, the Type III grout achieved higher temperatures in the smaller repair.

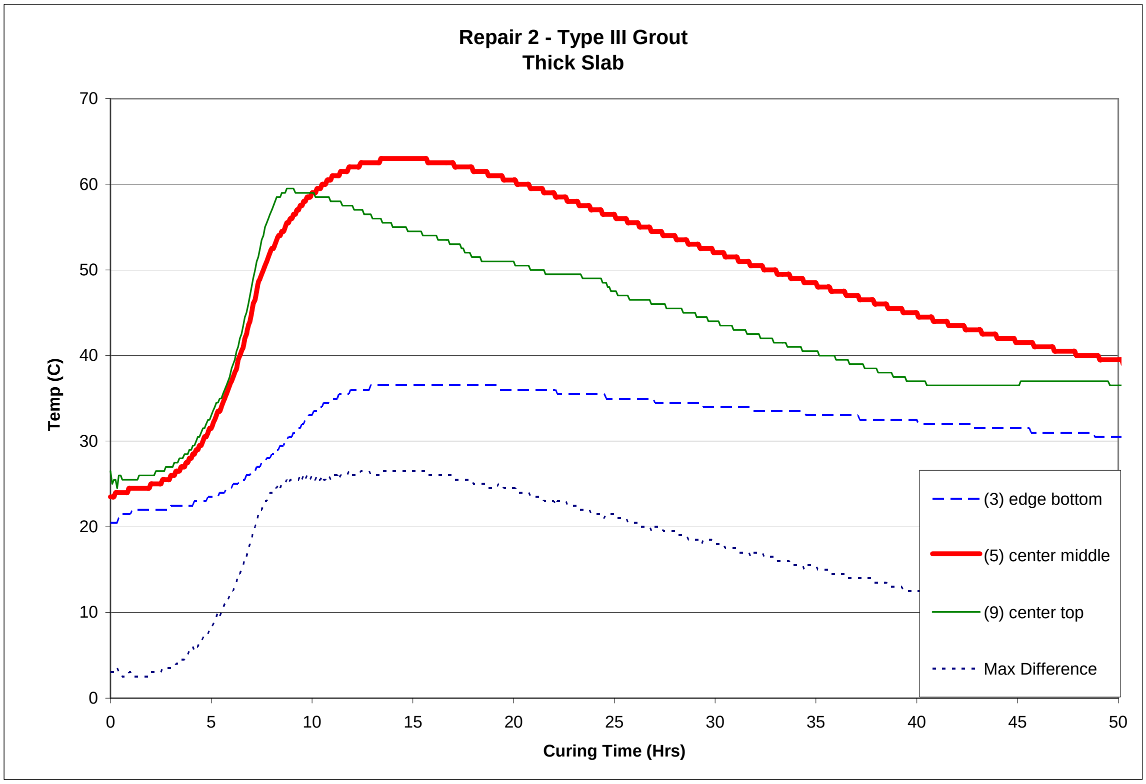

Figure 9. Temperature data for Repair 2

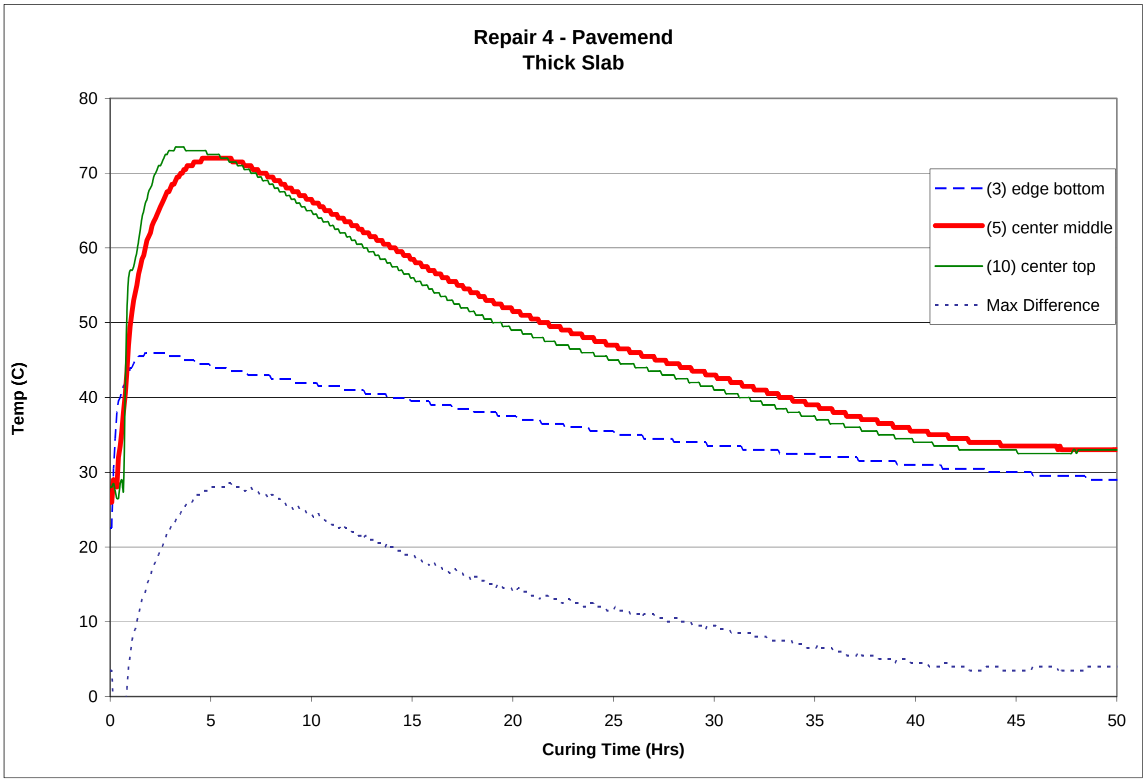

Figure 10. Temperature data for Repair 4

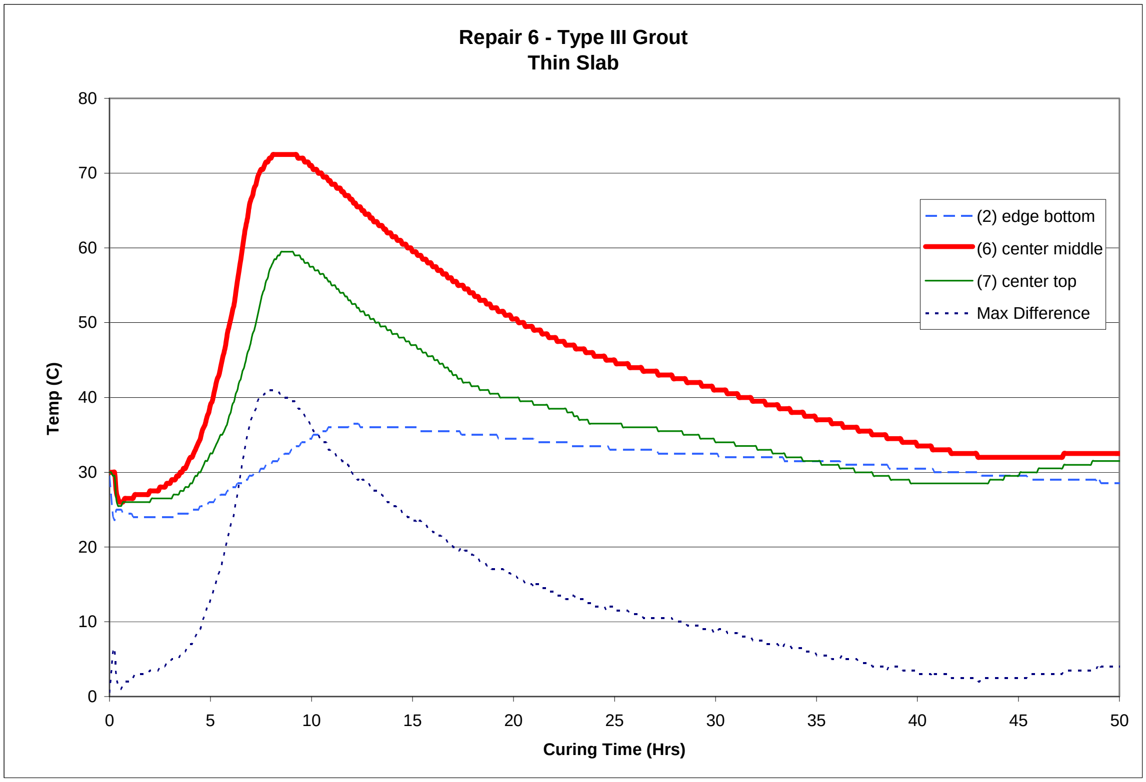

Figure 11. Temperature data for Repair 6

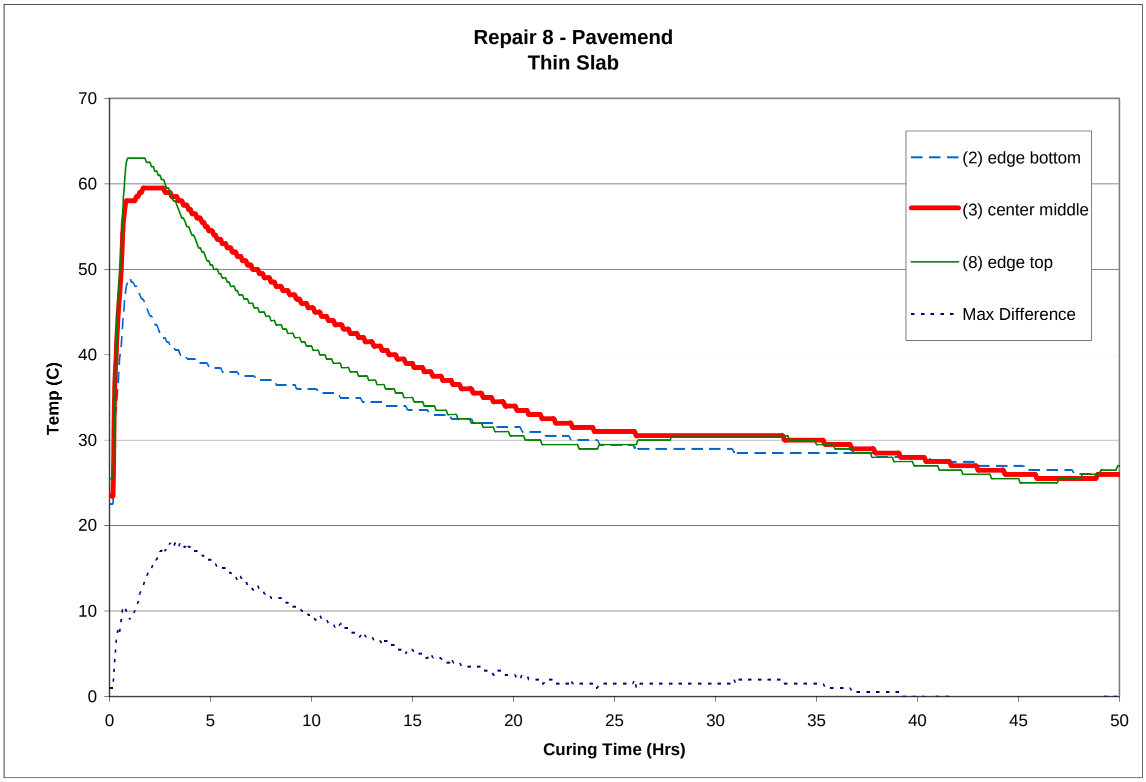

Figure 12. Temperature data for Repair 8

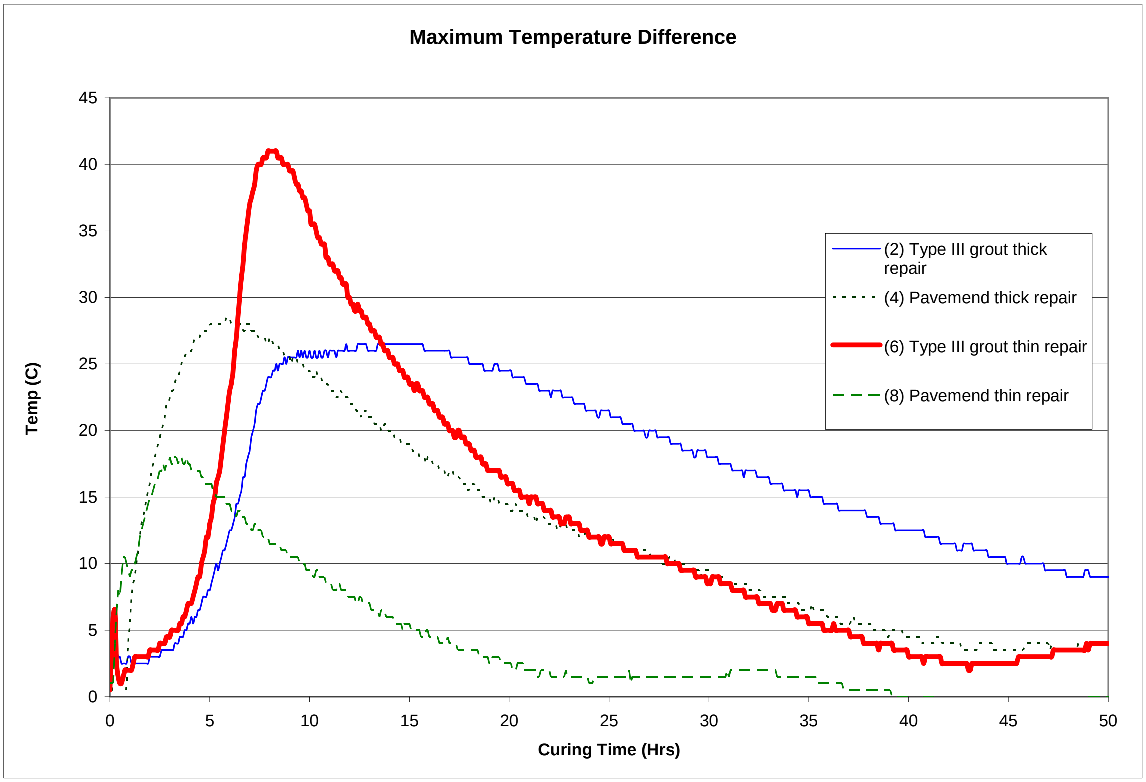

Figure 13. Summary of temperature differentials in Repairs 2, 4, 6 and 8

The maximum temperature differential occurred in Repair 6 at approximately 8 hours after placement. The thickness of the slab and volume of the material used does not appear to affect the maximum temperature or the maximum temperature differential. The maximum temperatures occurred in the middle of all repairs except for Repair 8, which peaked on the surface for a brief period of time; however, the middle remained the hottest portion for the majority of the recorded time.

Application of Traffic

The simulation traffic was designed for the purpose of validating the structural capacity of the repairs for the JRAC scenario. This includes relatively low pass levels of C-17 aircraft as the heaviest load. Traffic was applied to each repair using a military truck chassis that had been modified to serve as a load cart. The rear end of the load cart was equipped with a single C-17 tire and lead weights were arranged in order to create a single C-17 tire load of 20,000 kg (44,000 lb), as shown in Photo 33. The C-17 tire was inflated to an internal pressure of 0.98 MPa (142 psi) and the load was verified with a truck scale. A total of 50 passes of the single C-17 tire were applied to each repair. This validation process proposes that any repair method able to withstand (without substantial deterioration) the simulated C-17 traffic, which was applied at the end of a specified curing time, can therefore be considered as an appropriate repair technique for PCC pavements in the JRAC scenario. The Pavemend™ repairs were trafficked at 3 hours and the Type III portland cement grout repairs were trafficked at 24 hours. Photo 34 and Photo 35 show Repair 1 and Repair 3, respectively, after the application of 50 passes of the single-tire C-17 load cart. Photo 36 shows the application of traffic on Repairs 7 and 8.

Photo 33. Single-tire C-17 load cart applying 50 passes to Repair 3

All of the repairs were successfully trafficked (50 passes) and a visual inspection of the repairs after traffic revealed no significant damage to any of the repairs. Some minimal surface cracking was observed within 24 hours in both the Pavemend™ and Type III grout repairs around the outside of each repair at the interface with the parent slab; however, this cracking is not believed to be structural in nature.

Photo 34. Repair 1 after 50 passes of the C-17 load cart

Photo 35. Repair 3 after 50 passes with the C-17 load cart

Photo 36. Application of traffic on Repairs 7 and 8

Heavy Weight Deflectometer (HWD) Measurements

The repairs were evaluated using non-destructive testing (NDT) equipment to determine if the integrity of the repair was affected by the application of traffic. The NDT device used was a Dynatest® 8081 Heavy Weight Deflectometer (HWD), as described in Chapter 2. The trailer-mounted version of this device can be seen in Photo 37.

In this study the HWD was used for two purposes: to measure ISM of the repairs and to measure changes in deflection based LTE (Equation 1) between the repair and parent slab before and after traffic. The ISM was calculated by dividing the applied HWD load by the measured deflection at the center of the repair. The LTE was calculated by using the deflections under the load plate and the first sensor when they were positioned across a joint from each other. If a repair has deteriorated due to the repeated application of wheel loads, then a reduction would be expected in the LTE across one or more of the joints.

Photo 37. Conducting Heavy Weight Deflectometer (HWD) tests on Repair 8

HWD measurements were taken at three positions on each of the repairs before and after the application of traffic. The only exception was Repair 3, which did not receive pre-traffic measurements because the HWD was not available at the required testing time. The test positions were as follows: (1) the load plate on the parent slab with the first sensor located on the repair, (2) the load plate in the far edge of the repair with the first sensor on the parent slab, and (3) the load plate in the center of the repair. Position 1 was used to determine the load transfer efficiency from the parent slab to the repair. Position 2 was used to determine the load transfer efficiency from the repair to the parent slab. Position 3 was used to evaluate the impulse stiffness modulus of the repairs. The HWD was configured to produce an approximate load of 19,960 kg (44,000 lb) during the testing, which is equivalent to a single-tire load of a C-17 aircraft.

Load transfer efficiency was calculated using Equation 1 for two of the previously mentioned positions, both in the direction of the applied traffic. LTE data for Position 1 are shown in Figure 14. LTE data for Position 2 are shown in Figure 15.

The ISM was calculated using deflections measured at the load plate when the plate was positioned in the center of the repair. The ISM values were calculated before and after the application of traffic and the results are shown in Figure 16.

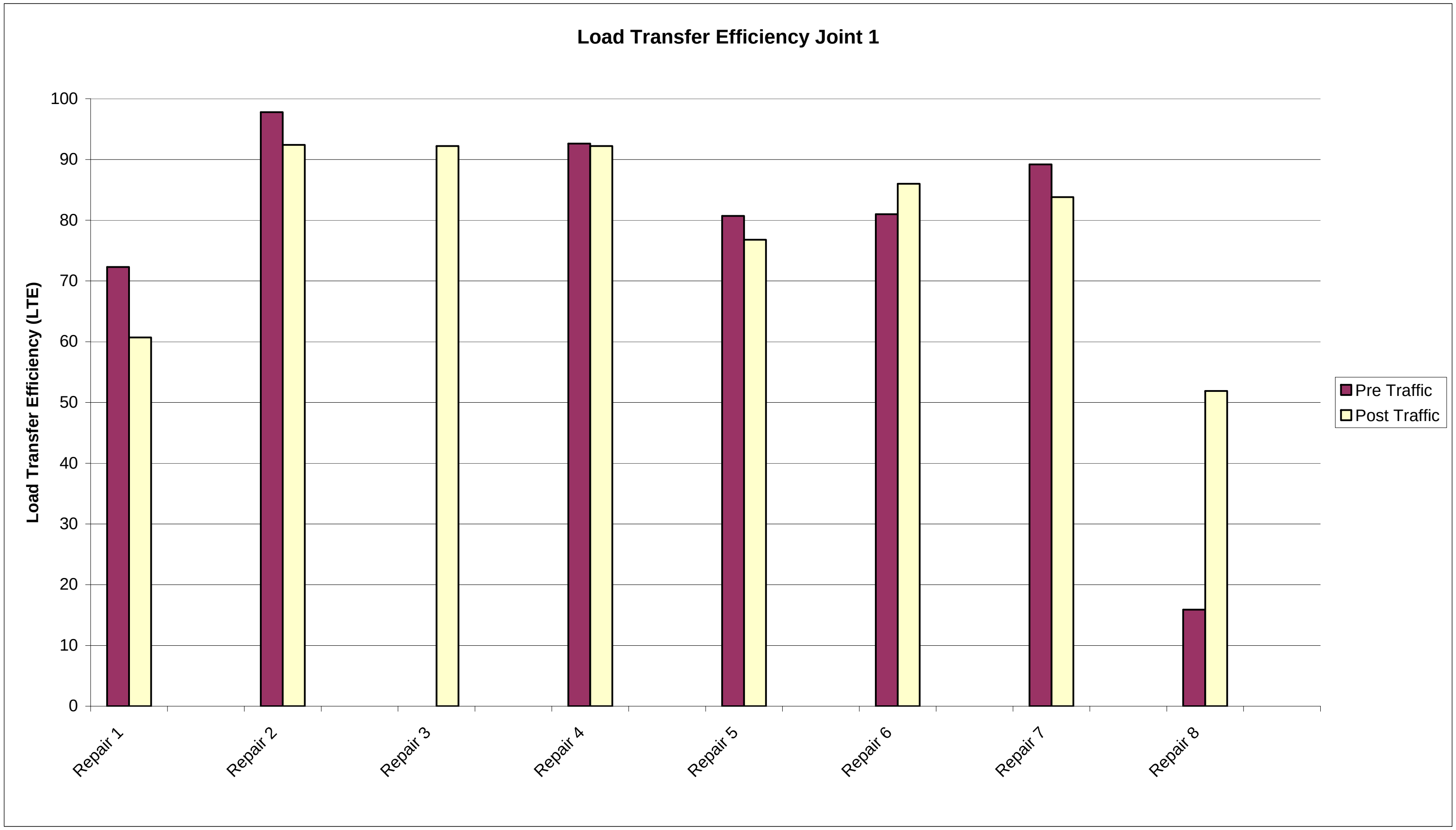

Figure 14. Load transfer efficiency for Position 1 (load plate on parent slab)

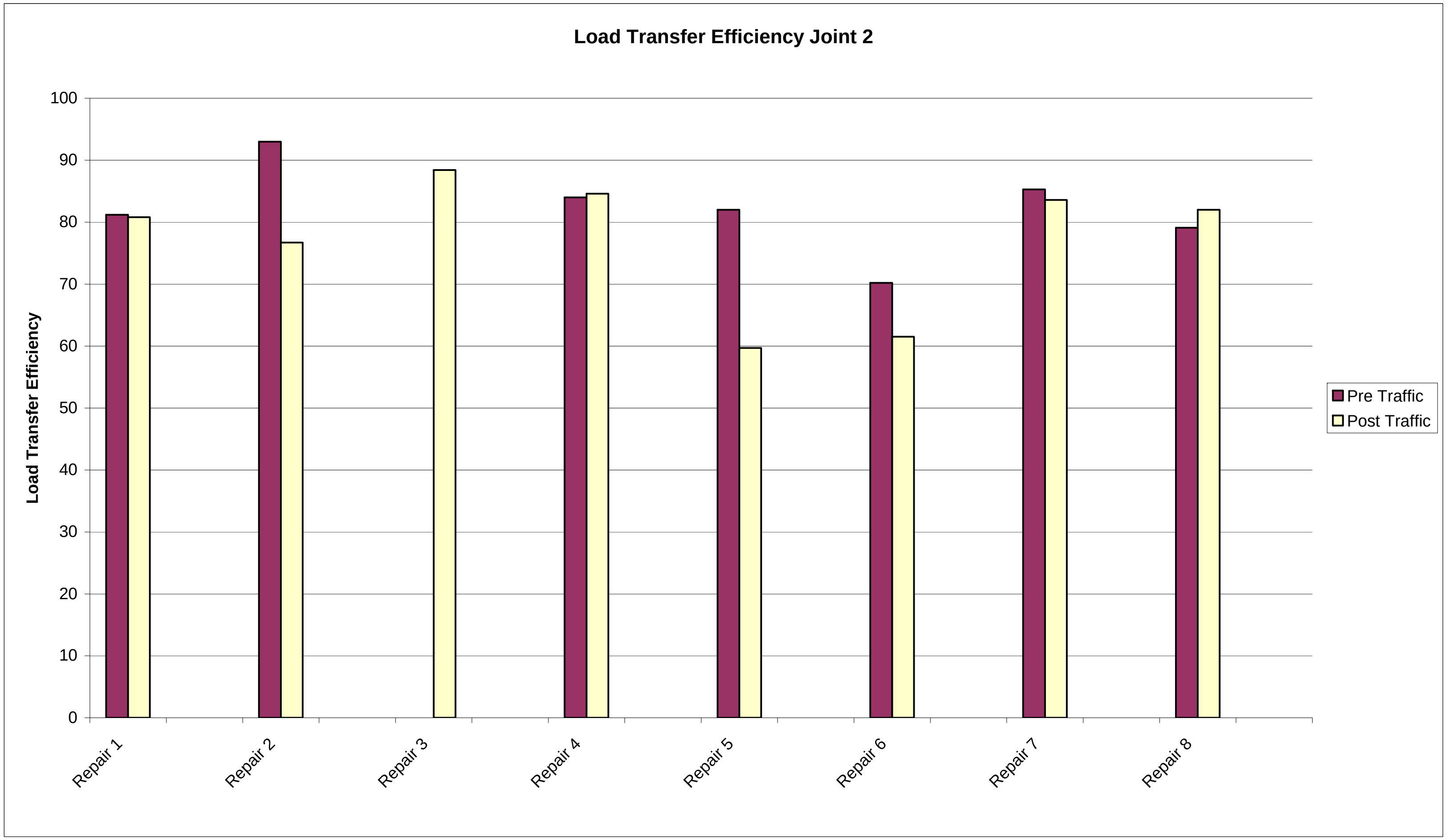

Figure 15. Load transfer efficiency for Position 2 (load plate on repairs)

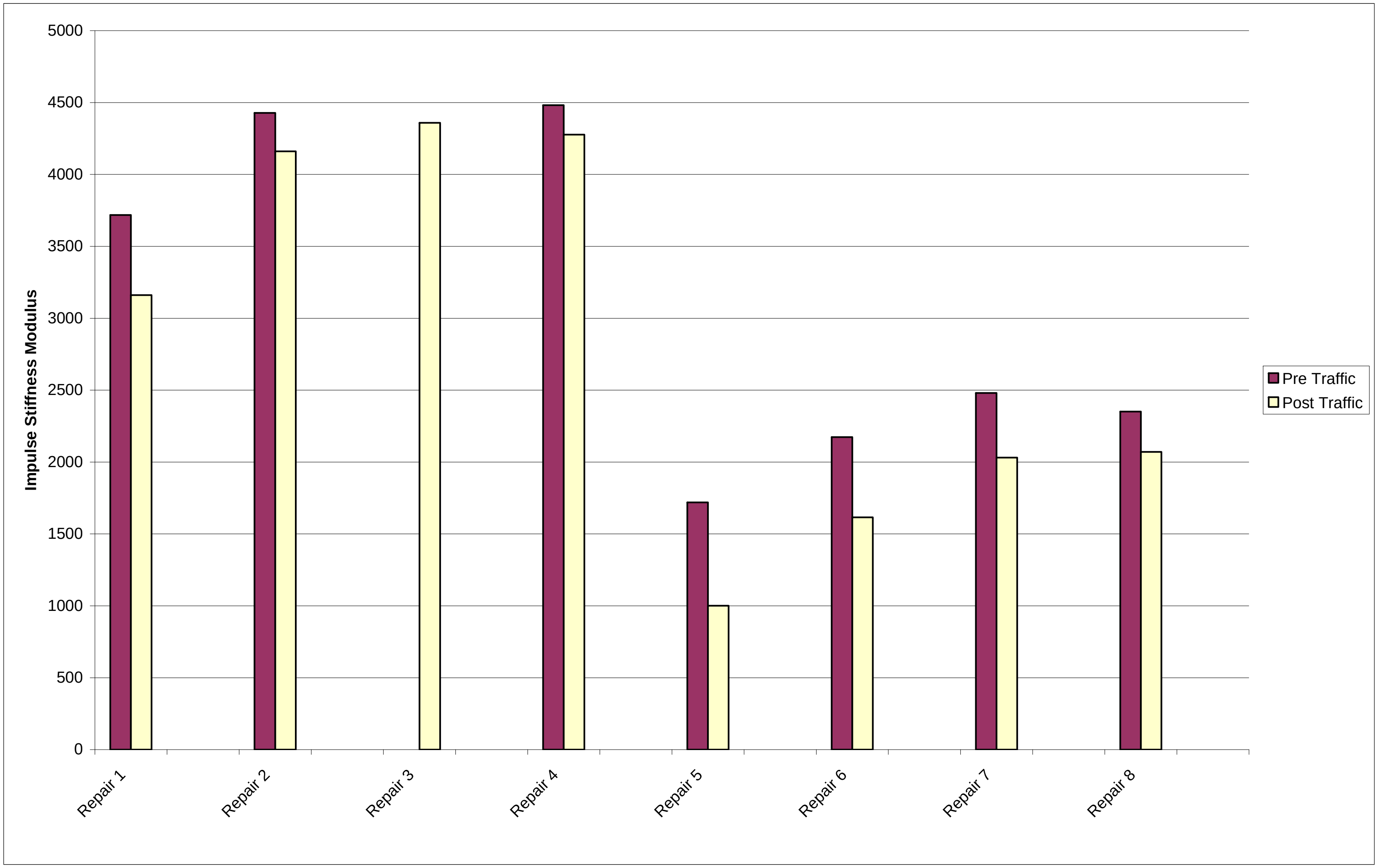

Figure 16. Impulse stiffness modulus (ISM) for repairs before and after traffic (Position 3)

The data from Position 2 provide more information and show larger differences between the repairs than Position 1, as the load plate didn't move the parent slab as much in Position 1. The largest percent change for Position 2 occurred in Repairs 2 and 5. Both consisted of Type III grout, and Repair 2 had a thickened edge while Repair 5 did not. The LTE and ISM data suggest that the thickened edge was more beneficial for the thinner slab and not necessarily required for the thicker slab. The thickened edge appears to benefit the Type III grout repairs more so than those made with Pavemend™. A reduction in LTE and ISM occurred in almost all of the Type III repairs after traffic; however, the effect on the Pavemend™ repairs was negligible.

The ISM was higher for Repair 2 with the thickened edge compared to Repair 1; however, the 102 mm (4 in.) of asphalt concrete surrounding Repair 1 makes it difficult to compare the two repairs. Traffic was placed on Repair 3 after 3 hours and there was not an opportunity to conduct pre-traffic HWD tests due to the equipment being unavailable. However, post traffic measurements were very similar to the results of Repair 4, indicating that a significant drop in any of the measurements would not be expected. LTE measurements on Repair 4 before and after traffic were essentially unchanged, indicating the traffic had very little affect on the repair.

Because of the large number of spall repairs located throughout the smaller slab, it was difficult to get the HWD plate and all the sensors on the slab without being affected by a previously placed spall repair. This is likely the cause of some of the erratic data (e.g., LTE on Repair 6 and 8).

Summary of Repair Performance

All of the repairs in the field portion of this study were successfully constructed and trafficked without failure. The construction techniques and mixing procedures were verified and all repairs were conducted within an acceptable timeframe for the JRAC program. The data indicated some losses in LTE and ISM after traffic, particularly in the thinner slab; however a visual inspection indicated no failures. Table 10 provides a summary of the repair characteristics.

The Type III grout repairs achieved much higher maximum temperatures and higher temperature differentials than expected. The Type III grout material got hotter and stayed hotter longer than the Pavemend™ repairs, which had steeper curves for heating and cooling. This would be expected because of the rapid nature of the reaction with Pavemend™.

The HWD tests were conducted just prior to and immediately following the application of traffic; therefore any significant changes can be contributed to the traffic and not other sources, such as thermal cracking. As expected, HWD test results indicated much higher ISM values for repairs conducted in the thicker slab. ISM results for the Type III grout repairs tended to be higher (before and after traffic) for those constructed with a thickened edge, however this was not the case for the Pavemend™ repairs. These data suggest that a thickened edge will benefit a repair made with Type III grout more so than one constructed with Pavemend™. The application of traffic resulted in higher reductions in ISM and LTE for the Type III grout repairs and had a lesser affect on the Pavemend™ repairs.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The research presented in this study was conducted to develop and validate a procedure for the rapid repair of intermediate-sized repairs on portland cement concrete (PCC) airfield pavements. The investigation included a laboratory portion to study the materials used in the procedure, including recycled concrete pavement (RCP), Pavemend™ grout, and a Type III portland cement grout. Full-scale field testing was also conducted to verify construction procedures, grout mixing and placing procedures, and structural validation for the design load.

The results of the study indicate that the proposed method of using RCP and flowable grouts provides an effective solution for the rapid repair of PCC airfield pavements while also minimizing the material quantities required to conduct the repair.

Laboratory

The results of the laboratory investigation yielded several conclusions regarding the materials used during the rapid repair tests. These conclusions include:

a. RCP provides a good source of aggregate for rapid repairs and greatly reduces the material quantities required compared to traditional techniques (about 50 percent savings). In this study, the system created by using RCP and grout achieved sufficient strength to meet the curing time objectives (3 or 24 hours, depending on type of cementitious material).

b. Pavemend™ can be effectively mixed in relatively large quantities using a mortar mixer; however, the 7-day and ultimate strength gains may be significantly less (as much as 20 percent) than if it is mixed using a hand drill and paddle mixer in single batches.

c. The measured thermal properties of Pavemend™ (specific heat, thermal diffusivity, and thermal conductivity) are all within the normal ranges of ordinary portland cement concrete.

d. Both Pavemend™ and the Type III grout provided full penetration of the selected gradation of RCP in the impregnation tests. The gradation used represents the minimum acceptable range and minimum particle size must be limited to 51 mm (2 in.). Maximum particle size should be limited to one-third of the total repair thickness.

Field Experiment

The construction of the repairs in the field environment resulted in numerous important conclusions. They include:

a. Construction of the repairs was accomplished with two sizes of wheel saws and hammers. The smaller equipment was found to be the most suitable for these types of repairs.

b. The time required to conduct each repair improved with experience and wasn't affected by the type of material used (Pavemend™ vs. Type III grout). The bulk unit weights of all materials determined previously provided an accurate tool to estimate material quantities required for each repair.

c. The thickened edge appears to benefit repairs made with Type III grout more so than those made with Pavemend™. Although successful, the repairs made without the thickened edge exhibited a reduction in ISM. If time allows, the thickened edge should be provided for all repairs.

d. A reduction in LTE occurred after traffic in all of the Type III grout repairs, however the effect on the Pavemend™ repairs was negligible. If time allows, the surfaces of the parent slab should be cleaned to improve bonding characteristics, especially when using Type III grout as the repair material.

e. Although there was no visual evidence of thermal cracking in any of the repairs, several of them had extremely large temperature differentials which could lead to thermal cracking and subsequent deterioration in repairs of larger size. Originally, the Pavemend™ repairs were expected to produce the largest temperature differentials; however, the data indicated that the Type III grout repairs produced the highest values.

f. A simple and easy-to-produce Type III portland cement grout is an effective alternative to high-priced repair materials if time allows for 24 hours of curing.

Recommendations

The purpose of this study was to investigate and develop new materials and techniques for the rapid repair of damaged PCC airfields using minimal material quantities and small, easy to transport, equipment. As such, a number of recommendations are made regarding the investigation:

a. The method of grout and RCP should be utilized by the military in contingency environments when rapid repair times are required for failed sections of PCC pavements.

b. Small, portable equipment such as that described in Chapter 4 is appropriate for intermediate sized repairs (approximately 1 m^3). Repairs of larger size are possible using these methods and materials, however they should be employed only if more long-term and permanent solutions are not available.

c. Constructions procedures including excavation, removal and processing of RCP, repair hole preparation, and grout mixing and placement, as described in Chapter 4, should be used to employ this technique.

d. Every effort should be made to ensure the quality of the repair, and the additional steps suggested in Chapter 4 should be taken if time and recourses allow. These include washing the RCP to remove dust and dirt, providing a thickened edge for repairs made with Type III grout, cleaning the surfaces of the parent slab, and moist curing for repairs made with Type III grout.

e. The methods and materials described in this report were evaluated under realistic field conditions, however only one ambient temperature range (optimum) was investigated. As such, extreme variations in actual temperature conditions must be fully considered and tested before repairs are made using these methods and materials.

Suggestions for Further Research

An investigation should be conducted to identify other materials which are suitable for this method of rapid repair. Field tests should be conducted to validate mixing procedures and compatibility with other types of RCP. These suggested efforts should be used to develop a list of approved products based on the type of application as well as guidelines for selecting the appropriate material.

Additional studies should be performed on Pavemend™ to determine the ultimate strength development with time and determine the cause of variability in results after 7 days.

Additional studies should also be conducted to develop an understanding of the negative effects of temperature for intermediate repairs of varying sizes. These studies should include recommendations for materials and methods for all expected temperature conditions which may be encountered during military contingency operations. Where appropriate, methods should be developed to effectively mitigate the negative effects which may include ways to dissipate heat from the center of the repairs to minimize temperature differentials and the potential for thermal cracking.

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14. ABSTRACT
The U.S. military must have the ability to rapidly deploy troops and equipment anywhere in the world as part of a contingency operation. Recent military operations have highlighted the critical need for rapid repair procedures and materials for military use on sub-standard, in-theater airfields. The U.S. Army Engineer Research and Development Center is currently addressing these problems through a 6-year demonstration-based research and development program called JRAC (Joint Rapid Airfield Construction). This study involves the development of a method using rapid setting grouts and recycled concrete pavement (RCP) to repair portland cement concrete pavements. A laboratory study was conducted to evaluate material properties in order to gain an understanding of expected field performance. Eight full-scale repairs were constructed using two rapid setting grouts, two types of equipment, and two concrete slabs. The repairs were successfully trafficked with simulated C-17 aircraft wheel loads to verify the structural capacity and, ultimately, the procedures.

15. SUBJECT TERMS

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